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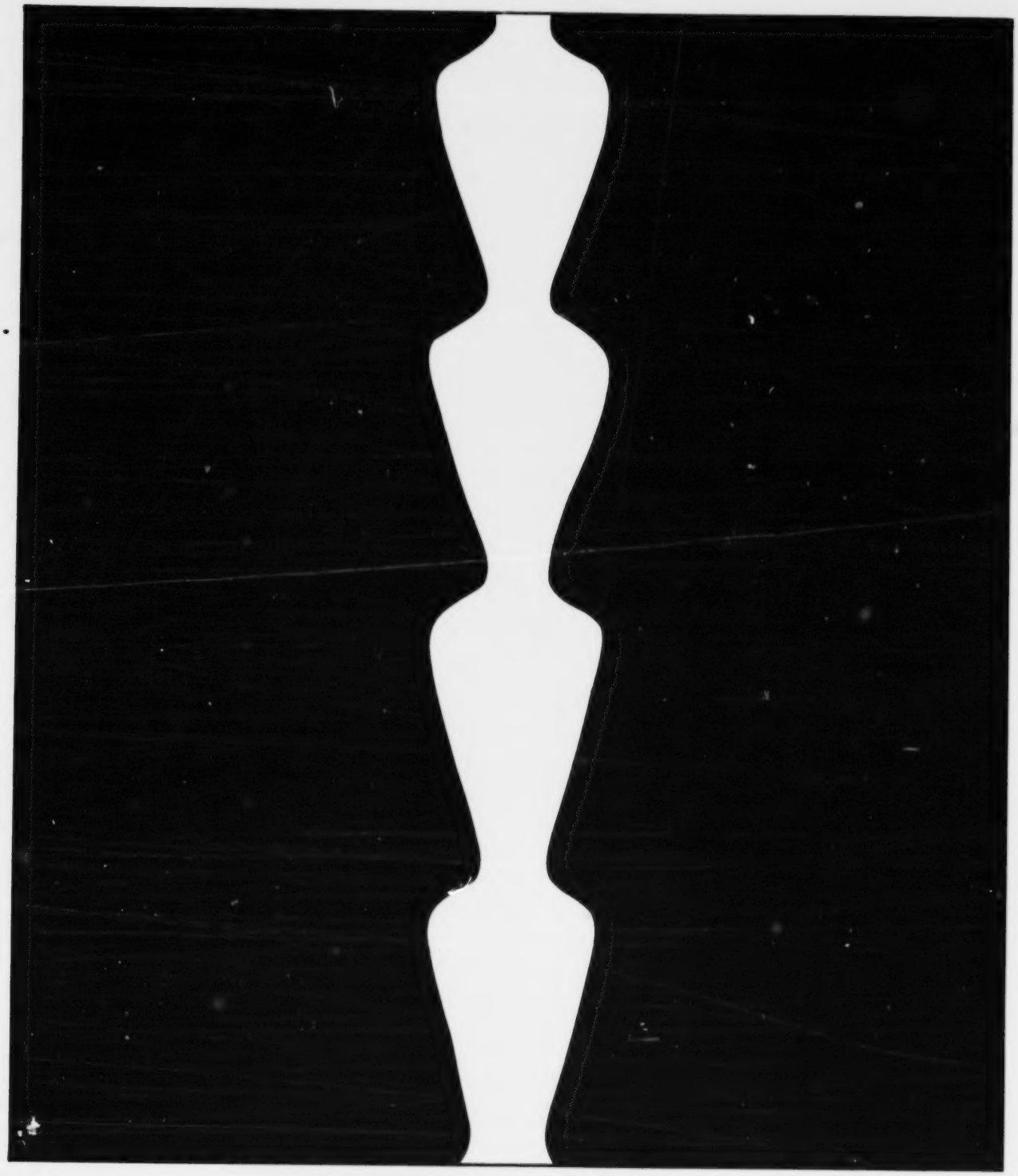
Road and Bridge Deck Drainage Systems



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Road and Bridge Deck Drainage Systems

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November 1982

Abstract

Hydraulic capacities of commonly used sewer inlets, ditch inlets, and bridge deck drains are not well known and this seriously impedes attempts to rationalize the design of road and bridge deck drainage systems. In this report, hydraulic capacities are given for a number of sewer inlets, ditch inlets, and bridge deck drains. For all these structures, hydraulic capacity curves were developed through full-scale laboratory testing under a wide range of design conditions. The susceptibility of ditch inlets and bridge deck drains to clogging by floating debris was also studied in the laboratory.

Using the experimental sewer inlet capacities, a new design procedure for inlet spacing was developed and demonstrated in two examples. Experimental drain capacities were used in the development of a new design procedure for bridge deck drainage. This procedure can be applied in an expedient computerized version.

Preface

This research study was undertaken under an agreement with the Province of Ontario to investigate road and bridge deck drainage systems on behalf of the Ministry of Transportation and Communications (MTC). The study has been conducted in three phases from 1977 to 1981 and the results were reported in three progress reports [8, 9, 10]. The report that follows is the final report on the study. The final report is not just a compilation of the progress reports but, wherever appropriate, additional analysis and interpretation of earlier results are presented.

Throughout this report, metric units are used except for some quotations from the literature. At the time of writing, there was no consensus on the choice of metric units for various quantities and, consequently, the author was advised by MTC to choose units as appropriate. The following choice of basic units was made:

- length —metres (m); millimetres (mm) for small dimensions
- area —hectares (1 ha = 10 000 m^2)
- slope —expressed in metres per metre (m/m)
- discharge—cubic metres per second (m^3/s).

Some additional comments regarding the above choice are given in the text. For the area units, considering the large areas one deals with and the current practice, hectares are used rather than m^2 as recommended in the International System of Units (SI).

As suggested by MTC, the slope was expressed as a direct value (i.e., the tangent of the slope angle, or the difference in elevation divided by the length), in metres per metre, as opposed to the values expressed in percents. Note that direct slope values are used in flow velocity and discharge formulae and the percentage slopes may lead to errors and misinterpretations. Also, the direct slope values are used predominantly in the Canadian and U.S. literature on road drainage. The expression of slope in percent seems to be limited to municipal drainage practice.

The discharge was expressed in m^3/s as opposed to sometimes-used *litres per second* (L/s) which is fairly common in European municipal practice. The decision to use m^3/s was based on the fact that it is the unit recommended in the SI system and, among the metric units, it is used almost exclusively in the scientific and technical literature dealing with hydraulics and hydrology.

Acknowledgements

The author gratefully acknowledges the assistance received from many staff members of the Ministry of Transportation and Communications. In particular, special thanks are due to Mr. J. E. Gruspier who was the technical liaison officer throughout the whole study and to Messrs. D. G. Guibord, J. D. Harris, and R. A. Madill, who actively participated in various stages of the study.

Finally, the author would like to acknowledge the invaluable help received from the staff of the Hydraulics Division of the National Water Research Institute in Burlington. Messrs. D. Doede, F. Dunnett, and Mrs. R. Purves conducted laboratory experiments, Mrs. B. Jones prepared the text, and Drs. T. M. Dick and Y. L. Lau reviewed the study report.

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1 / Introduction

High costs of drainage facilities prompted the Ontario Ministry of Transportation and Communications (MTC) to undertake a review of their drainage design practices. This review indicated that with a better knowledge of the behaviour of various drainage elements, the design procedures could be further rationalized and the drainage costs reduced. Consequently, the Ministry commissioned the Hydraulics Division of the National Water Research Institute in Burlington to undertake a study of road and bridge deck drainage systems. Although the primary functions of both systems are quite similar, there are some important distinctions and, whenever required, the two systems are discussed separately.

Road drainage consists of surface drainage elements, sewer inlets, and conveyance elements which are either pipes or ditches. Sewer inlets play a key role in road drainage because they affect both the rate of water removal from the road surface and the degree of utilization of the conveyance elements. Insufficient inlet capacity may cause road flooding hazardous to traffic and it may also lead to under-utilization of the sewer pipes downstream of the inlet. Overdesign of inlets results in too many costly inlet structures. It is obvious that efficient design of road drainage cannot be achieved without an accurate knowledge of sewer inlet hydraulics.

Several investigations of sewer inlets have been reported in literature but, in general, those studies do not satisfy the needs of the Ministry because the data apply to inlets different from those which are currently used by MTC. Note also that there is a trend in municipal stormwater management to reduce the inflow to storm sewers by restricting inlet capacities and increasing water ponding on the street surface. While such practices may be acceptable under some municipal conditions, they seem to contradict MTC concerns for traffic safety. In newly designed systems, the designer can judiciously control the division of runoff flow between the surface and subsurface drainage elements by proper inlet spacing.

Bridge deck drainage has received little attention in the past, because it rarely leads to a structural failure. Consequently, many regard drainage as an inescapable nuisance, rather than a problem. It should be recognized, however, that poor drainage leads to a number of costly problems which could be avoided by a proper design.

Among the problems caused by poor drainage are ponding, uncontrolled drainage discharges, and cold weather problems. Ponding has a variety of causes and adversely affects traffic as well as the bridge structure. Uncontrolled drainage discharges may cause erosion, settlement of pavement slabs, and even structural failure. The runoff water falling on the bridge structure may cause stains and discolouration of exposed faces if it is not collected and disposed of properly. Runoff may also wash off corrosive contaminants and bring them into contact with structural members, perhaps causing deterioration of the bridge structure. Other drainage problems are brought about by cold weather. The freezing of infiltrated water can cause considerable damage.

It is obvious from the above examples that one cannot over emphasize the importance of getting all the liquids off the bridge, in a controlled manner, and as soon as possible. From the drainage point of view, it would be desirable to space drains quite closely to reduce the length of travel of water on the bridge and, consequently, to reduce the possibility of drain clogging. When deciding on the spacing of drains, however, the designer has to consider the cost of drain installations and find a compromise between the cost of drainage and the nuisance or damages arising from underdesigned drainage facilities. It should be borne in mind that the cost of bridge deck drains varies substantially. In open country, a drain can be just an inexpensive piece of pipe installed in the deck. On the other hand, in urban conditions, typical drains are more elaborate structures accompanied by piping.

A literature review has not revealed any information on actual hydraulic capacities of deck drains. Consequently, there is no rationale for designing the drain spacing.

The present report attempts to remedy the lack of data for the design of road and bridge deck drainage systems and to improve the existing design procedures. The terms of reference of the study described here were presented elsewhere [5, 7, 13] and could be summarized as follows.

- Determine the hydraulic capacities of existing and new inlets, for selected design conditions, by means of full-scale testing in the laboratory.
- Determine the hydraulic capacities of commonly-used bridge deck drains by means of full-scale testing in the laboratory.
- Evaluate the susceptibility to blockage of selected ditch inlets and bridge deck drains. If feasible, recommend improvements in grate designs.
- Develop design procedures for spacing of sewer inlets and bridge deck drains on the basis of laboratory results.

The last task (design procedure) requires considerations of two additional drainage aspects—the stormwater runoff computations and gutter flow. Both subjects are briefly reviewed in the following two chapters.

2/ Stormwater Runoff

The computation of stormwater runoff was not included in the terms of reference of the study reported here. Consequently, only a general discussion of the procedures currently employed by MTC is given here for completeness. For details, the reader is referred to the MTC *Drainage Manual* [3].

Assuming the applicability of the Rational Method to road drainage, runoff can be related to rainfall intensity by the following formula:

$$Q = C\,i\,A \qquad (1)$$

where Q is the peak runoff rate, C is a runoff coefficient depending on characteristics of the drainage area, i is the average rainfall intensity for the selected frequency and the time of concentration, and A is the drainage area.

In common metric units, Equation 1 has the following form:

$$Q\ (m^3/s) = 0.00278\ C\ i\ (mm/h)\ A\ (ha) \qquad (2)$$

where 0.00278 converts the units.

For paved surfaces such as King's Highways and freeways, the runoff coefficient is typically taken as C = 0.95 and the concentration time which affects the selection of the intensity varies from 10 to 15 min.

When designing road drainage, the designer typically assumes a 10-year design storm, and the corresponding rainfall intensity is obtained from the local intensity-duration-frequency curves.

The Rational Method has some inherent limitations which have been discussed in detail elsewhere [3, 11]. It suffices to say, for the purpose of this report, that the use of the Rational Method implies a steady-state flow in gutters and through the inlet grates. Since the method neglects the attenuation of runoff peaks by road-surface storage, it probably overestimates runoff peaks. It should be stressed, however, that the experimental inlet capacities presented later in this report are applicable in conjunction with any runoff computations.

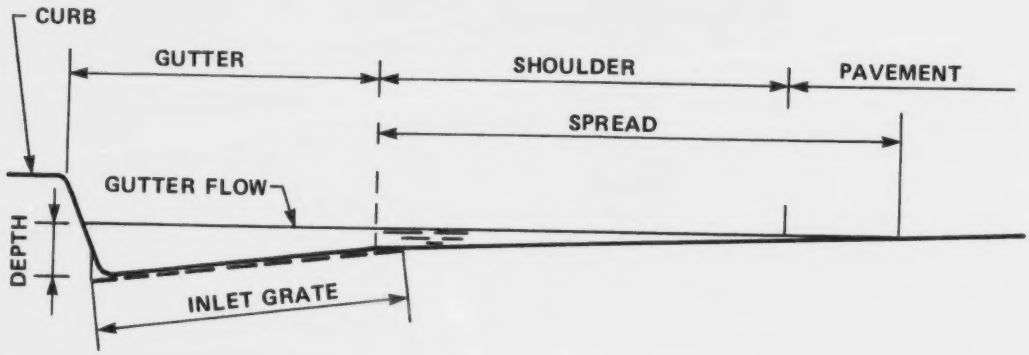


Figure 1/ Cross-Section of a Road Drainage Channel

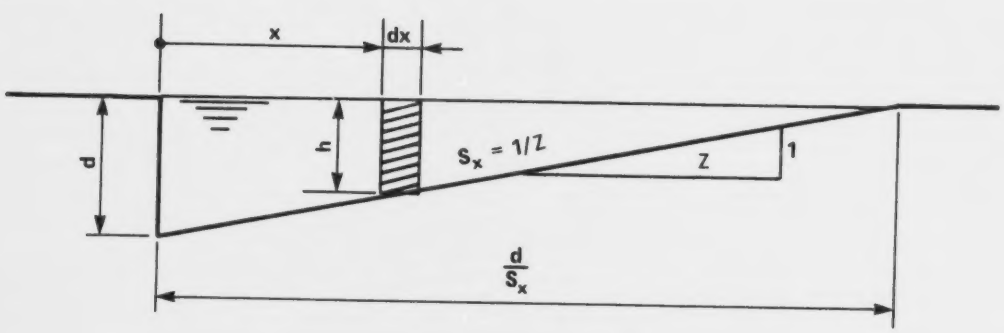


Figure 2/ Notation Sketch — Flow in a Triangular Channel

3/ Gutter Flow

For the purpose of this report, the flow in the channel formed by the curb, gutter, shoulder, and the pavement is referred to as the *gutter flow* (see Figure 1). The channel geometry shown in Figure 1 is not universal, because some roads are designed without gutters (i.e., the deep narrow channel next to the curb).

The most comprehensive treatment of gutter flow was offered by Yen [18] who used the St. Venant equations to describe the unsteady flow in gutters. For practical reasons, the current road drainage design is based on the Rational Method and, consequently, gutter flow is assumed to be steady. Under such an assumption, any of the flow friction formulae can be applied to calculation of the gutter flow. The Manning formula was found to be used widely [15] and was, therefore, adopted in this study.

In metric units, the Manning equation reads as follows:

$$v = \frac{1}{n} R^{2/3} S^{1/2} \qquad (3)$$

where v is the flow velocity (in m/s), n is the Manning roughness coefficient (taken here as n = 0.013 according to the terms of reference), R is the hydraulic radius (R = A/O, where A is the flow area and O is the wetted perimeter), and S is the longitudinal slope (grade). The gutter flow rate Q is calculated from the continuity equation as:

$$Q = v A \qquad (4)$$

Thus, the capacity of a gutter depends on its cross-section, grade, and roughness.

Some previous search [6] indicated that the Manning equation cannot be used without modification to compute flows in shallow gutter sections because the hydraulic radius does not adequately describe the gutter cross-section. This becomes particularly obvious for large spreads which may be about 50 times larger than the flow depth at the curb. Consequently, Larson [6] recommended the application of the Manning equation to a small increment of the channel width and the integration of the incremental flow rate dQ over the full width of the cross-section.

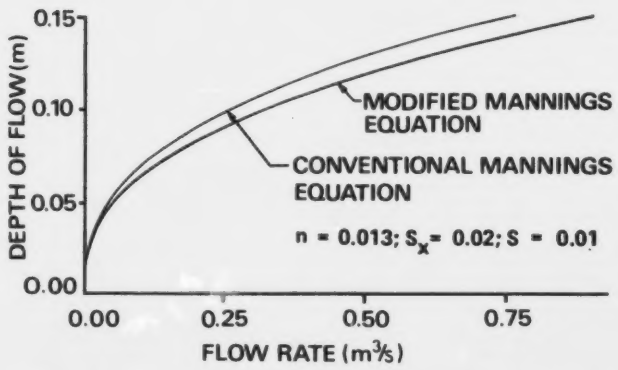


Figure 3/ Flow Rating Curve of a Triangular Gutter

Referring to Figure 2, the aforementioned integration is shown below for the simplest gutter geometry (a triangular cross-section).

For an incremental area dA = hdx, the discharge dQ can be calculated as:

$$dQ = vdA \qquad (5)$$

where the velocity v can be expressed from the Manning equation as:

$$v = \frac{1}{n} R^{2/3} S^{1/2} \qquad (6)$$

The hydraulic radius for the incremental area can be written as:

$$R = \frac{hdx}{dx} = d - x S_x \qquad (7)$$

After substituting Equations 6 and 7 into Equation 5, one obtains:

$$dQ = \frac{1}{n} S^{1/2} (d - x S_x)^{5/3} dx \qquad (8)$$

Integration of Equation 8 over the full flow width yields:

$$Q = \frac{0.375}{n S_x} S^{1/2} d^{8/3} \qquad (9)$$

where S is the gutter grade, S_x is the crossfall, and d is the depth of channel, at the deepest point. It is of interest to note that, if one applies Equations 3 and 4 to a triangular channel, the final expression for Q is identical to Equation 8 except for the numerical constant of 0.315. Thus, the modified (integrated) Manning equation yields flow rates, for triangular gutters, about 19% larger than those calculated from a conventional application of the Manning equation. As an example, discharge curves for a triangular gutter (n = 0.013, S_x = 0.02, and S = 0.02) were calculated by both methods and are shown in Figure 3.

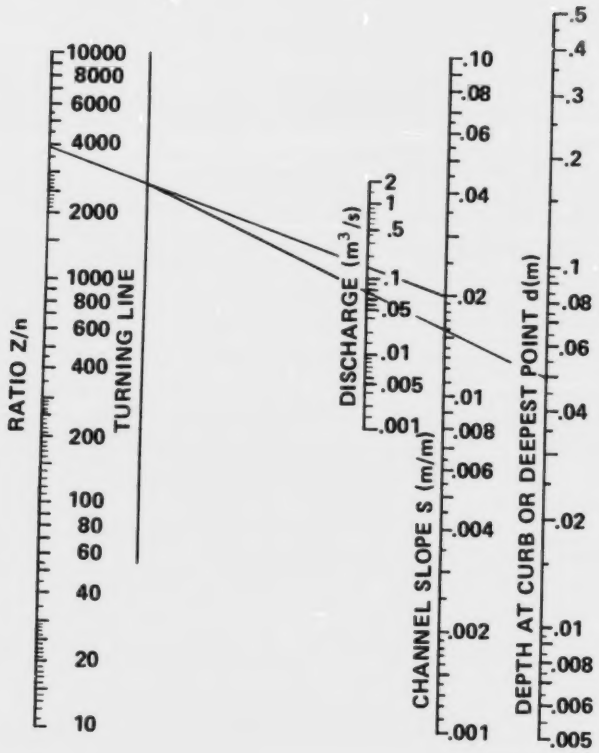
As an alternative to numerical calculations of the discharge in a triangular gutter, one may use the nomograph in Figure 4.

An approach similar to that described above was used to derive flow discharge curves for all the cross-sections studied and the longitudinal grade varying from 0.003 m/m to 0.10 m/m. These cross-sections are shown in Figure 5. The discharge curves are presented later as design charts (Section 8.2).

EQUATION

$$Q = 0.375 \frac{Z}{n} S^{1/2} d^{8/3}$$

Q = triangular channel discharge
Z = reciprocal of crossfall
n = Manning's roughness coefficient
S = channel slope; d = flow depth



EXAMPLE

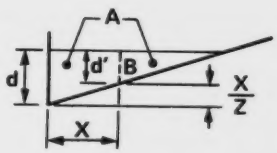
Given: S = 0.02, Z = 50, n = 0.013, Z/n = 3846, d = 0.05 m
Find: Q = 0.070 m^3/s

INSTRUCTIONS

1. Connect Z/n ratio with slope S; intersect with turning line connect with depth d to obtain discharge Q.

2. For shallow V-shaped channel shown below use Nomograph with Z=T/d.

3. To determine discharge Q_x in portion of channel having width x: determine depth d for total discharge in the entire section A. Then use Nomograph to determine Q_B for depth. $d' = d - \frac{X}{Z}$



4. To determine approximate discharge in composite section: Follow instructions 3 to obtain discharge in section A at assumed depth d; obtain Q_B for slope ratio Z_B and depth d', then $Q_T = Q_A + Q_B$

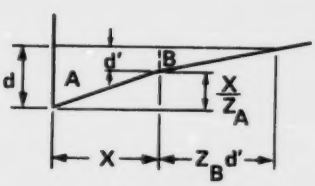


Figure 4/ Nomograph for Flow in Triangular Channels
(Based on Chart By U.S. Federal Highway Administration)

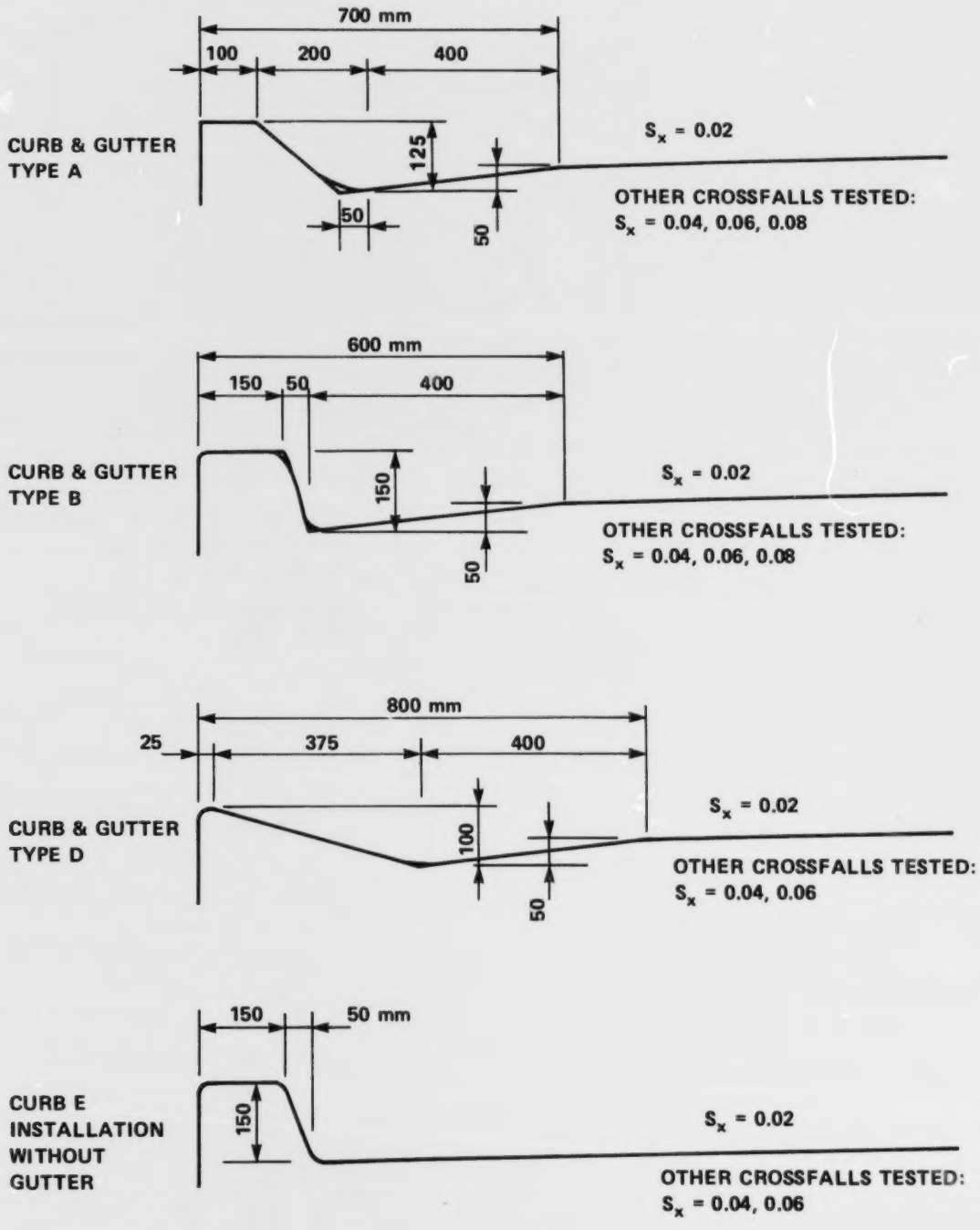


Figure 5/ Cross-Sections of Tested Curbs and Gutters

4/ Hydraulics of Sewer Inlets

Sewer inlets can be divided into three major classes, each with many variations. These classes are: *grate inlets, curb inlets,* and *combination inlets.* Considering the use of various types of inlets by MTC, grate inlets installed on a grade and in a sag were studied most extensively. For comparative purposes, one combination inlet installed on a grade was also studied. Curb inlets were excluded completely from the study.

Inlet capacities can be determined by different methods. Among the approaches reported in the literature, scale model studies and numerical approximations are particularly common [12, 16, 19]. Both approaches have some inherent limitations. In model studies, it is difficult to achieve hydraulic similitude because of small flow depths and spacings between the grate bars. Analytical approaches require numerous approximations which lead to large uncertainties in the calculated inlet capacity. Consequently, the Ministry requested the establishment of inlet capacities through a full-scale laboratory testing. A description of these tests follows.

4.1/ Experimental Apparatus

A special apparatus for testing inlets was built in the Hydraulics Laboratory of the National Water Research Institute. This apparatus, which is shown in Figure 6, consisted of the following principal components:

- a water supply tank,
- a roadway section with an inlet, and
- a water return structure.

The water supply tank delivered water to the roadway and also served for metering the gutter flow. The tank was 2.75 m wide, 3.66 m long, and 0.91 m deep. At the downstream end of the tank, a suppressed rectangular weir was placed. The weir was calibrated *in situ* and the rating curve had the following form:

$$Q\ (m^3/s) = 1.89\ (0.85 - 0.1h)h^{1.512}(m) \qquad (10)$$

where Q is the weir discharge, and h is the weir head in metres. The weir head was measured by a point gauge placed inside of a stilling well attached to the tank. The inflow rates to the experimental roadway were then calculated from Equation 10.

The water supply tank was connected to the roadway by a wooden chute. The chute had a bottom slope of 0.40 m/m and its width increased from 1.2 m to 2.4 m over a length of 0.7 m. To reduce the velocity of water leaving the supply tank, the chute was covered with several layers of a horse-hair mattress and fitted with several baffles. This arrangement produced satisfactory inflow conditions at the upstream end of the experimental roadway.

The experimental roadway was built on a wooden platform which was 4.20 m wide and 12.0 m long. The platform was supported by three I-beams which rested on 12 pillars. Using mechanical jacks, the platform longitudinal and crossfall slopes could be easily changed. In early tests of inlets on a grade, the longitudinal slope was varied from 0.003 m/m to 0.0964 m/m in the following steps: 0.003, 0.01, 0.02, 0.03, 0.04, 0.06, 0.08, and 0.0964 (m/m)

The crossfall slopes were varied from 0.02 m/m to 0.08 m/m, in 0.02 increments. In later stages of the study, the maximum grades and crossfalls were both 0.06 m/m.

For the tests of inlets in a sag, vertical wooden barriers were built along the perimeter of the wooden platform. Although the terms of reference specified the maximum depth of water as 0.20 m, the facility was allowed to reach depths in excess of that limit (up to 0.30 m).

On the top of the platform, road drainage channels were installed. All the parts were made of plywood. To simulate properly the channel roughness, sand particles with a 1.5 mm diameter were glued to the wood surface. This particle size was found to correspond, for a fully-developed turbulent flow, to the Manning coefficient n = 0.013.

A square grid with fine divisions of 0.25 m and coarse divisions of 1 m was marked on the experimental roadway. This grid served for setting-up gutter flows of preselected pavement spreads.

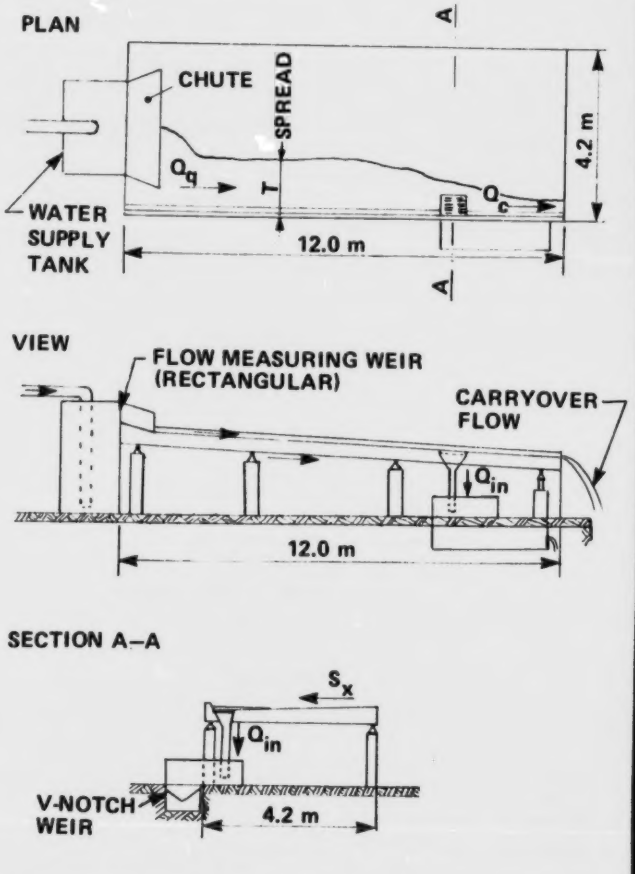


Figure 6/ Experimental Apparatus for Testing Inlets

The tested sewer inlets were installed about 8 m downstream from the upstream end of the roadway. The inlets were installed according to MTC specifications [14]. Water entering the inlet discharged into a stilling basin connected to a weir box serving for flow measurement. The measuring weir was a 90° V-notch weir for which the following rating curve was derived from laboratory tests:

$$Q\ (m^3/s) = 1.38\ h^{2.529}(m) \qquad (11)$$

where Q is the weir discharge and h is the weir head in metres. The weir head was measured by a point gauge installed in a stilling well.

In sag installation tests, all roadway flow discharged through the inlet. In installations on a grade, water was leaving the experimental roadway in two ways—either through the inlet, or as a carryover flow discharged at the downstream end of the facility. The carryover flow was collected below the downstream end of the roadway and discharged into the laboratory sump.

4.2/ Experimental Procedures

Individual tests were preceded by setting up the roadway longitudinal and crossfall slopes. This setting was done manually by adjusting the platform supports and checking the roadway levels with an automatic levelling instrument. Typically, the actual surface elevations did not deviate from the calculated ones by more than 1 mm.

For each setting of longitudinal and crossfall slopes, inlet capacities were measured for several discharges through the roadway drainage channel, starting with small flows and increasing them gradually until the pavement spread of 3 m was reached, or the drainage channel was filled to the top of the curb. During each test run, the gutter flow rate, spread and depth (upstream of the inlet), and the inlet discharge were measured.

Both the gutter and inlet discharges were measured fairly accurately by the calibrated measuring weirs. It is estimated that the accuracy of flow measurements was about 3% to 5% at the readout point.

The flow spreads were measured upstream of the inlet where the waterline was more or less parallel to the road curb. The accuracy of these measurements is estimated as $\pm$ 0.05 m.

The measurements of the flow depth were complicated by the presence of surface waves (which also occur in actual field conditions).

4.3/ Grate Inlets on a Grade

Grate inlets are used most frequently in road drainage. They can be installed flush with the plane of the normal gutter (undepressed inlets), or they can be depressed below the normal gutter plane (depressed inlets). Both installations have some advantages and disadvantages. Undepressed inlets are cheaper to install and interfere less with traffic than the depressed ones. Depressed inlets have larger hydraulic capacities than undepressed inlets.

When setting research priorities for the study reported here, investigations of both undepressed and depressed inlets were considered. A preliminary analysis of experimental inlet capacities indicated that the typical design flows specified by MTC for the inlet spacing of 150 m could be accommodated, in most cases, by the tested undepressed inlets. Consequently, the depressed inlet installations were not studied.

Apart from testing the selected existing inlets in both single and twin installations, the feasibility of developing new grate designs of improved capacity was studied in a special test series.

The hydraulic capacity of a grate inlet on a grade depends upon its geometry and upon the characteristics of the gutter flow. In fact, on flat grades the inlet capacity is more controlled by the gutter flow discharge than by the inlet geometry. Under typical conditions, an inlet intercepts only a portion of the gutter flow and the rest bypasses the inlet as a carryover flow.

4.3.1/ Experimental Programme

The scope of the grate inlet testing programme was established by MTC in consultation with the Hydraulics Division. As suggested by MTC, the required number of experiments was reduced by interpolations and extrapolations of observed data. Such procedures had to meet the accuracy criterion specified by MTC—the inlet capacities were to be determined with an accuracy of $\pm$ 10%.

The whole programme comprised more than 220 runs and close to 1200 measured points. A summary of the experimental programme is given in Table 1.

4.3.2/ Experimental Results and Discussion of Results

The results obtained for various grate inlets are presented in the following order:

- Popular MTC inlets —DD-713-A, DD-713-B, DD-705, Twin DD-713-A, Twin DD-705
- Two municipal inlets—KWC, Round Frame
- New designs —Design Nos. 1-10

4.3.2.1/ Grates DD-713-A and DD-713-B, Curb and Gutter A

The dimensions of the curb and gutter type A are shown in Figure 5, the inlet grates DD-713-A and DD-713-B are shown in Figure 7.

The inlet grate (bars parallel to the curb) was tested for a full range of conditions as specified by MTC (see Table 1). Observed inlet capacities are shown in Figure 8 and listed in the Appendix.

The observed inlet capacities increased rapidly with increasing longitudinal slope up to a certain point beyond which the inlet capacity remained nearly constant. In the region of small longitudinal slopes, the inlet capacity is clearly controlled by the gutter flow which limits the supply of water to the inlet. With the increasing gutter flow, the inlet capacity increases up to a point where the flow velocity interferes with the inflow into the inlet and this inflow remains more or less constant, or is even reduced for the steepest slopes.

If the inlet capacity is considered as a fraction of the gutter flow rate, then the highest values would be obtained for low longitudinal slopes and small spreads where the inlet capacity may equal the gutter flow rate ($\bar{Q}_i = Q_{in}/Q_g = 1$, all the gutter flow is intercepted). With increasing grades and spreads, a smaller and smaller fraction of the gutter flow is intercepted. In fact, for the steepest grade and the maximum spread, less than 10% of the gutter flow is intercepted by the inlet. Such a finding agrees in general with results published previously by others [16].

As expected, inlet capacities increased with the increasing spread and crossfall. Both these conditions result in an increased depth of flow over the inlet grate. For larger crossfalls, the gutter flow is not only deeper at the curb, but it is also more concentrated (lower spreads).

Subsequently, grate DD-713-B (the bicycle grate) was tested in the same installation. The results of these tests are plotted in Figure 9 and listed in the Appendix. A brief examination of Figure 9 shows that grate DD-713-B behaves similarly to grate DD-713-A. A detailed comparison of capacities of grates DD-713-A and DD-713-B indicated that, on the average, the capacity of grate DD-713-A was about 8% higher than that of grate DD-713-B. The standard deviation of the ratio $R = Q_{713A}/Q_{713B}$ was 0.08 and about three-quarters of all observed capacities were within the range $1.0 < R < 1.16$. In several cases, the capacities of grate DD-713-A were larger by 30% than those corresponding to grate DD-713-B. These largest deviations could be affected by experimental errors.

Furthermore, the differences between both inlet capacities were examined with regard to the grade, crossfall, and spread. For steep grades and large crossfalls and spreads, grate DD-713-A capacities were significantly larger than those observed for grate DD-713-B. Under those conditions, grate DD-713-A induced less disturbance to the flow over the inlet than grate DD-713-B.

Table 1/ Grate Sewer Inlets on a Grade, Testing Programme

Curb and Gutter Type	Inlet Grate Type	Grade (m/m)	Crossfall (m/m)	Spread* (m)
A	DD-713-A	0.003, 0.01, 0.02, 0.03, 0.04, 0.06, 0.08, 0.0964	0.02, 0.04, 0.06, 0.08	0.5 m to 3 m (without overtopping the curb)
A	DD-713-B			
B	DD-713-A			
B	DD-713-B			
D	DD-705	0.003, 0.01, 0.02, 0.03, 0.04, 0.06	0.02, 0.04, 0.06	
E	DD-713-B			
(No gutter)				
B	Twin DD-713-A			
D	Twin DD-705			
B	KWC**			
B	Round-Frame	0.003, 0.02	0.02	
B	New Designs	0.003, 0.02	0.02	

*The spread was measured from the upper edge of the gutter (i.e., the edge closer to the pavement — see Figure 1). In installations without gutters, the spread was measured from the curb.

**Kitchener-Waterloo-Cambridge grate.

In both cases, the crossbar (perpendicular to the curb) in the middle of the grate seemed to disturb the flow pattern. Fast flowing water hitting the crossbar was deflected upwards and partly jumped over the downstream half of the grate.

When comparing the total areas of openings of the two discussed grates, it was noted that the grate DD-713-A had the total opening area about 7 % larger than the grate DD-713-B. Under these circumstances, grate DD-713-B seemed to perform quite well. It appeared that the bars oriented at a 45° angle to the curb helped to concentrate the flow closer to the curb, thus creating larger depths of flow over the grate and increasing the inlet capacity. At high velocities, the crossbar caused the flow to partly jump over the grate.

On low grades (< 0.01 m/m), there was hardly any difference in the capacities of both grates. For steeper grades, grate DD-713-A offered larger capacities and would allow larger inlet spacings.

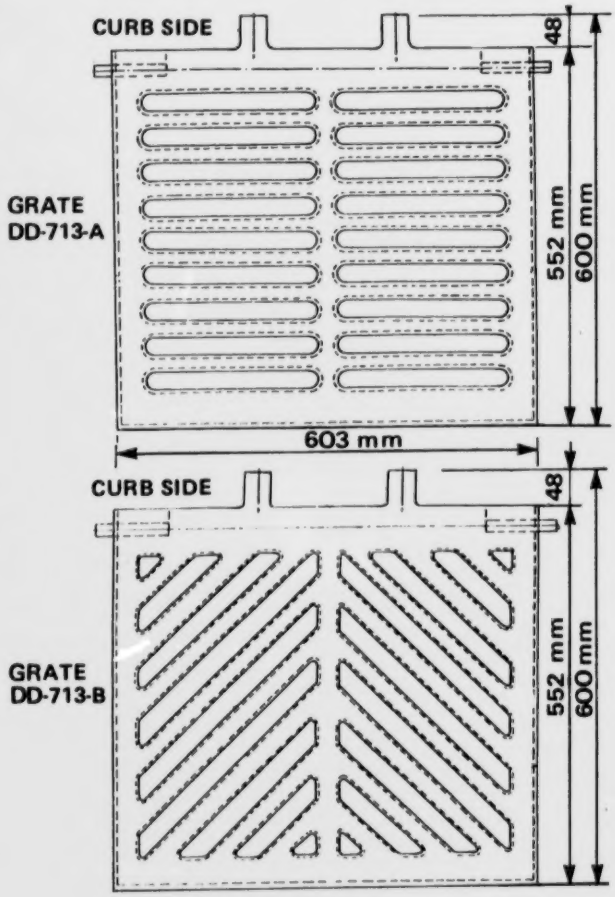


Figure 7/ Inlet Grates DD-713-A and DD-713-B

4.3.2.2/ Grates DD-713-A and DD-713-B, Curb and Gutter Type B

The dimensions of the curb and gutter type B are shown in Figure 5. In relation to the type A, type B has a slightly smaller cross-sectional area (for identical depths of flow) and, consequently, a smaller flow capacity. One would expect, therefore, that the inlet capacities for the type B should be also smaller than those for the type A, particularly in the region where the inlet capacity is most affected by the gutter flow rate. It was also noted that the average depth over grates installed in conjunction with the curb and gutter type B is slightly smaller than that for the type A.

Experimental data confirmed the expected behaviour of inlets installed with the curb and gutter type B (see Figures 10, 11 and the Appendix). When comparing mean inlet capacities for all the grades, crossfalls, spreads, and grates, the inlet capacities for the curb and gutter type B were 4% smaller than those for the type A.

Curb B has one advantage in comparison to curb A —it allows greater depths of flow (by 0.025 m) and, consequently, larger pavement spreads can be achieved without overtopping the curb.

Inlet capacities of grates DD-713-A and DD-713-B were compared for curb and gutter type B. On the average, the grate DD-713-A had capacities larger by 9% than the grate DD-713-B. This finding is consistent with that reported earlier for curb A.

Inlet capacity curves for the curb and gutter type B were similar to those observed for the type A. This fact was utilized in the interpolations of the experimental data.

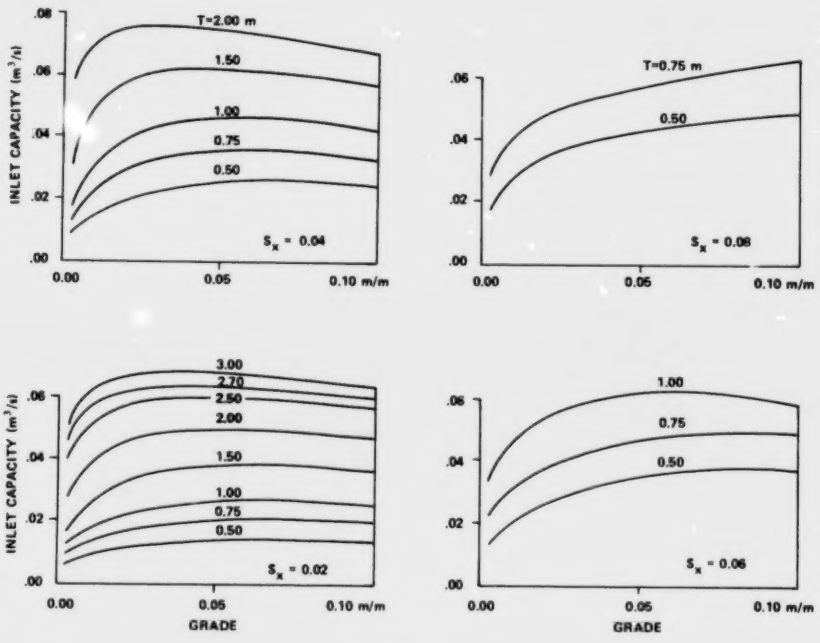


Figure 8/ Sewer Inlet Capacity: Curb & Gutter Type A, Inlet Grate Type DD-713-A

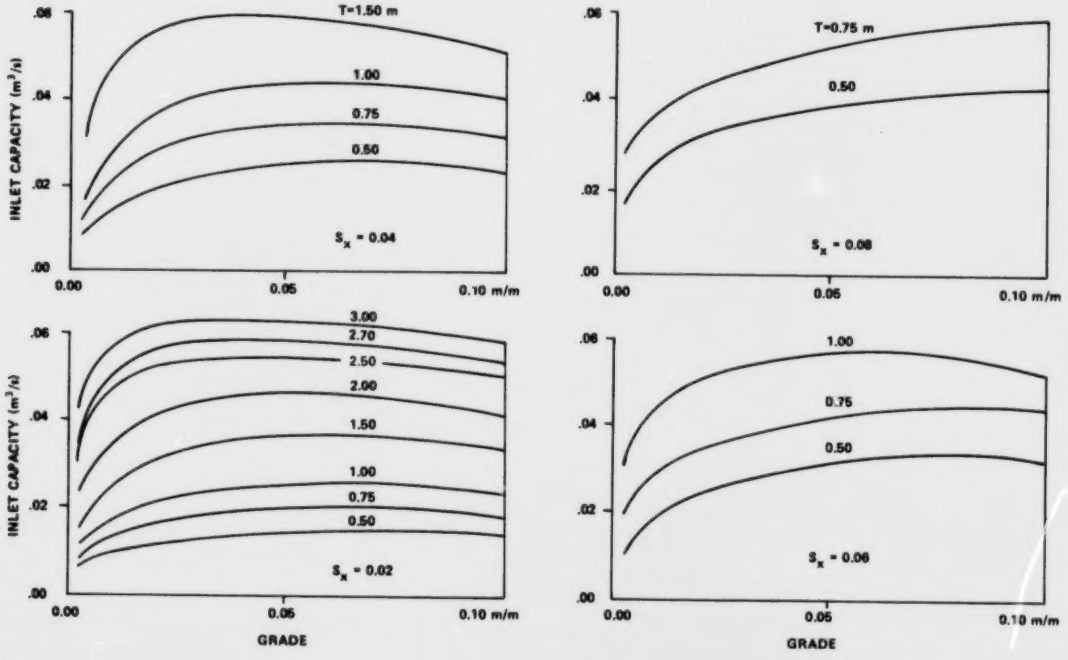


Figure 9/ Sewer Inlet Capacity: Curb & Gutter Type A, Inlet Grate Type DD-713-B

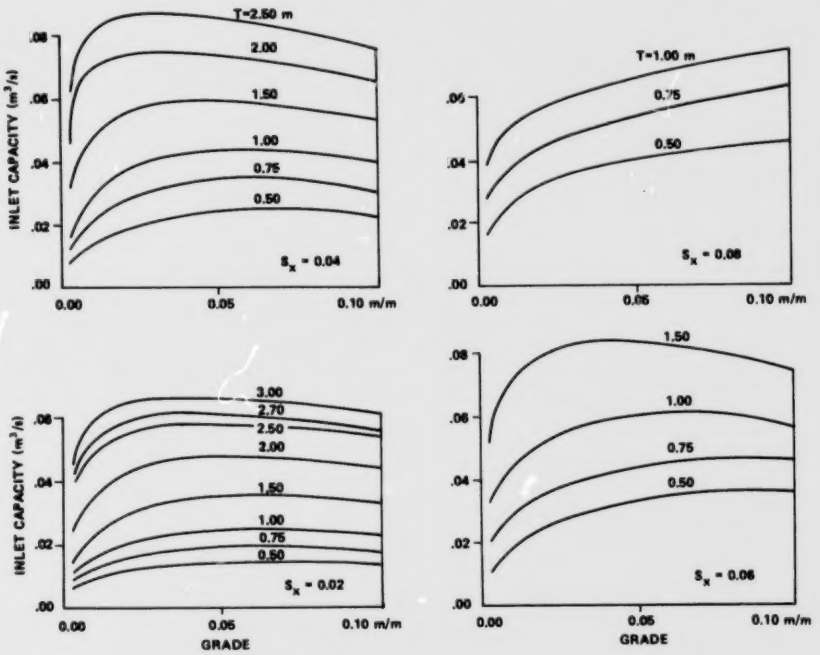


Figure 10/ Sewer Inlet Capacity: Curb & Gutter Type B, Inlet Grate Type DD-713-A

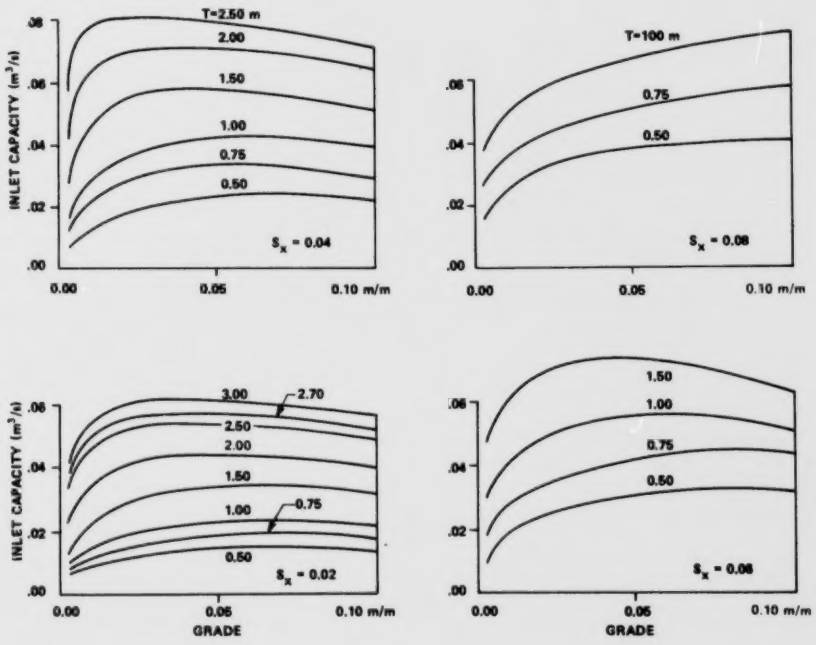


Figure 11/ Sewer Inlet Capacity : Curb and Gutter Type B, Inlet Grate Type DD-713-B

4.3.2.3/ Grate DD-705, Curb and Gutter D

The curb and gutter type D (see Figure 5) is frequently used for drainage of freeways and differs substantially from the previously discussed types. The gutter channel has a V-shape, with a slope 4:1 (i.e., horizontal:vertical) on the curb size and the slope 8:1 on the pavement side. The height of the curb measured from the deepest point in the gutter to the top of the curb is only 100 mm. Consequently, the range of pavement spreads which can be achieved without overtopping the curb is somewhat limited.

Grate DD-705 is used in conjunction with the curb and gutter type D (see Figure 12).

The capacity curves observed for grate DD-705 are shown in Figure 13 (see also data listings in the Appendix). The shape of these curves is similar to that of the capacity curves discussed earlier. In the region of small longitudinal slopes ($S_o < 0.02$ m/m), the inlet capacity was very much affected by the gutter flow rate. The inlet capacity increased rapidly with an increasing longitudinal slope. Outside of the discussed region (i.e., for $S_o > 0.02$ m/m), the capacity curves became fairly flat with more or less a constant discharge through the inlet. In some cases, the inlet capacity even decreased with increasing longitudinal slope and increasing gutter flow velocities.

Other observations made with regard to grate DD-713-B (crossbar effects, inlet efficiency) applied to grate DD-705 as well.

Because gutter type D is relatively shallow ($d = 100$ mm), the maximum inlet capacities and pavement spreads achieved without overtopping the curb were significantly lower than those reported for curb types A and B.

4.3.2.4/ Grates DD-713-A and DD-713-B, Curb E (no gutter)

Roadways without gutters (i.e., without the narrow deep channels along the curb) are sometimes used in urban areas because of space limitations.

Inlet grates DD-713-A and DD-713-B are both used in the drainage of roadways without gutters. To reduce the study costs, only grate DD-713-B was tested, assuming that the data for grate DD-713-A could be confidently extrapolated from the data obtained for grate DD-713-B.

The dimensions of the roadway without gutters are shown in Figure 5. The inlet capacities observed for grate DD-713-B are shown in Figure 14 and the capacities extrapolated for grate DD-713-A are shown in Figure 15. Inlet capacities are also listed in the Appendix.

The shape of the inlet capacity curves in Figure 14 is similar to that observed for other curb and gutter arrangements. For identical spreads, the roadway without gutters had flow depths over the inlet grate smaller than those found in the previous tests of roadways with gutters (types A and B, see Figures 8 and 9). Consequently, the inlet capacities for roadways without gutters are smaller than those presented earlier for the curb and gutter types A and B.

When extrapolating capacities for grate DD-713-A, it was assumed that the grate DD-713-A capacities are larger than those for grate DD-713-B. The data in Figures 8 to 11 were used in these extrapolations.

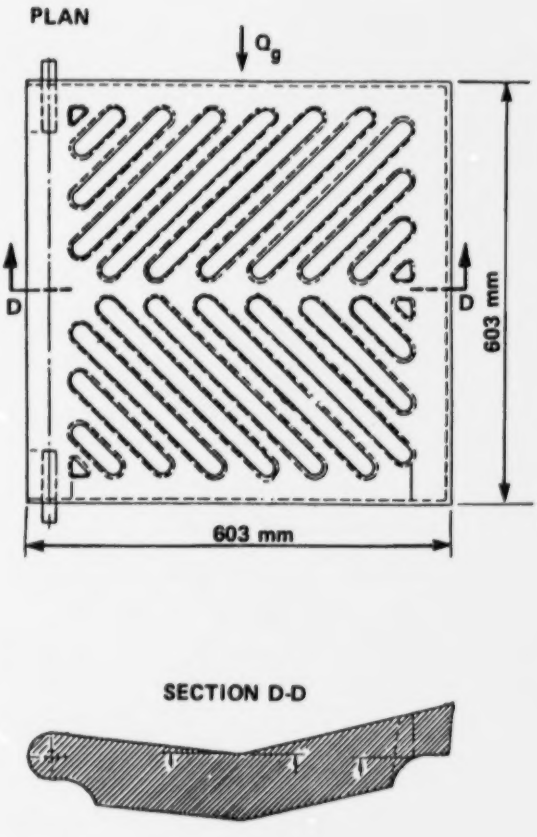


Figure 12/ Inlet Grate DD-705

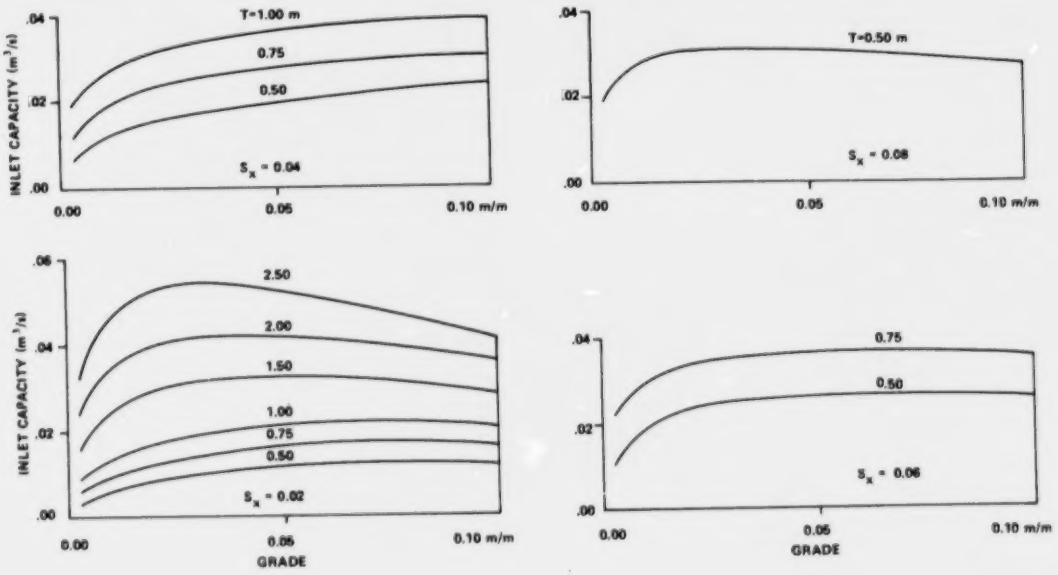


Figure 13/ Sewer Inlet Capacity: Curb & Gutter Type D, Inlet Grate Type DD-705

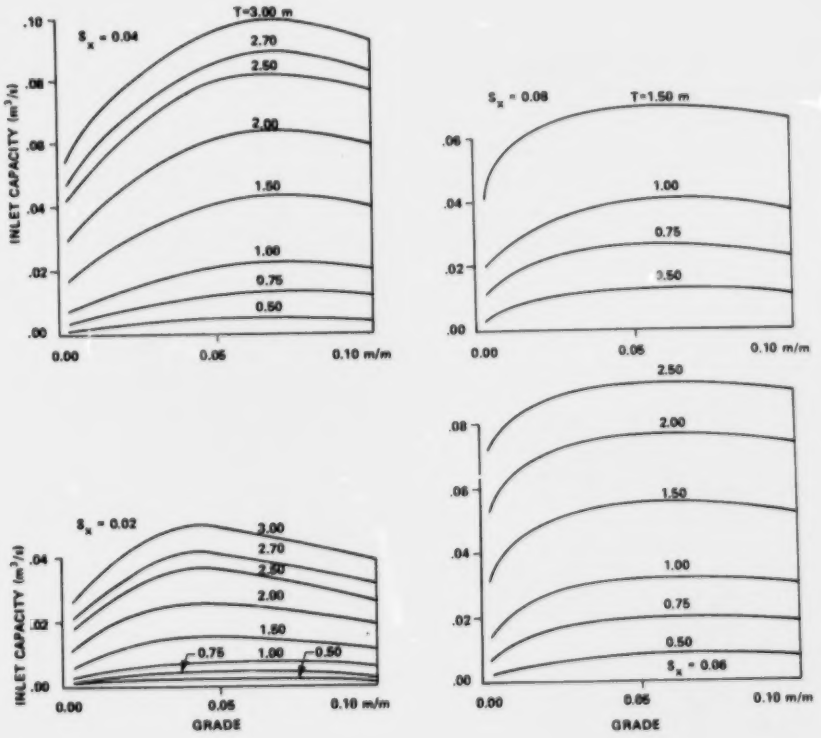


Figure 14/ Sewer Inlet Capacity: Curb E (no Gutter), Inlet Grate DD-713-B

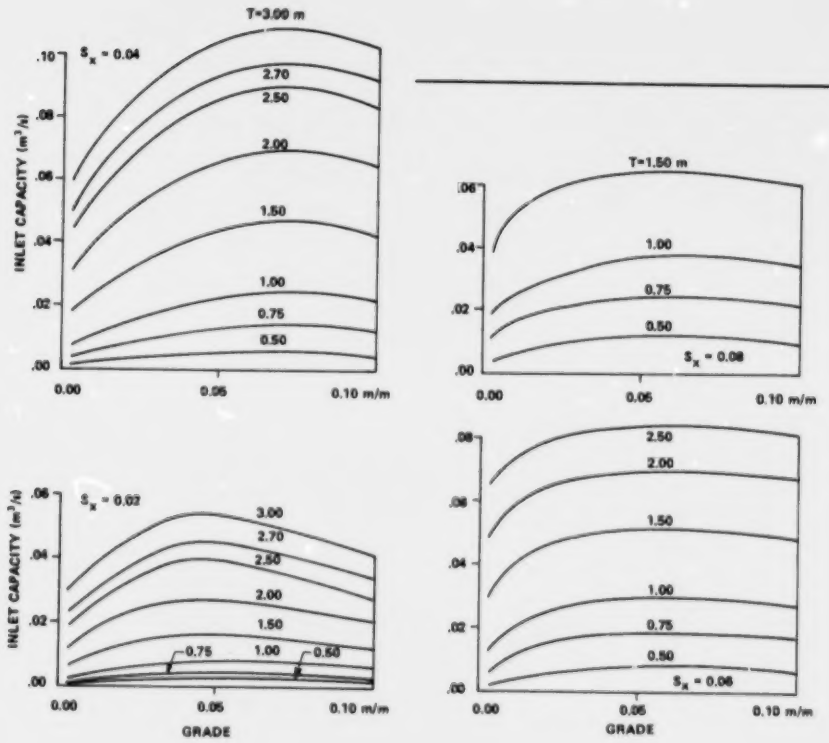


Figure 15/ Sewer Inlet Capacity: Curb E (no Gutter), Inlet Grate DD-713-A

Table 2/Calculated Capacities of Single and Twin Inlets

Line No.	Calculated Quantity	Numerical Values in Columns No. 1-6 Were Calculated for Various Assumed Efficiencies of Single Inlets					
1	Assumed single inlet efficiencies ($E = Q_{in}/Q_g$)	1.00	0.80	0.60	0.40	0.20	0.05
2	Single inlet discharge in fractions of the gutter flow	1 00	0.80	0.60	0.40	0.20	0.05
3	Carryover flow = $Q_g - Q_{in}$	0.00	0.20	0.40	0.60	0.80	0.95
4	Discharge through the second grate, in fractions of the gutter flow, assuming the same efficiencies as in line No. 1 (line 1 x line 3)	0.00	0.16	0.24	0.24	0.16	0.05
5	Total discharge through the twin inlet (line 2 + line 4)	1.00	0.96	0.84	0.64	0.36	0.10
6	Twin inlet discharge / Single inlet discharge (line 5 ÷ line 2)	1.00	1.20	1.40	1.60	1.80	2.00
	Repeating the above calculation but assuming that the second grate efficiency is only one half of the values assumed for the first one.						
7	Twin inlet discharge / Single inlet discharge	1.00	1.10	1.20	1.30	1.40	1.50

4.3.2.5/ Twin Grates DD-713-A and DD-705

Under special circumstances, all the standard inlets discussed so far can be installed as twin inlets to increase the inlet capacity. In such installations, the inlets are installed in series with the spacing between the inlets varying from 0.23 m to 0.30 m. In the installations studied here, the spacing was 0.30 m measured between the inside edges of frames (see Figure 16). Parallel twin inlets (i.e., two inlets side-by-side extending from the curb towards the road centre) have been proposed in the past [16] but rejected because of interference with traffic.

Twin inlets installed in series do not obstruct traffic, but their efficiency is typically smaller than that of parallel inlets. In a twin inlet installation, the second inlet intercepts relatively small flows for the following reasons.

- Some fraction of the gutter flow is intercepted by the first grate; thus, the inflow to the second grate may be largely reduced, particularly in the vicinity of the curb.
- For high flow velocities, the first grate causes flow disturbances which will reduce the hydraulic effectiveness of the second grate.

The first point is illustrated in Table 2, which shows theoretical capacities of single and twin inlets for assumed hydraulic efficiencies.

It appears from Table 2 that, although twin inlets have capacities larger than single inlets, the increases will rarely exceed 50% of the single inlet capacity.

The economic feasibility of using twin inlets as opposed to single inlets deserves a closer examination. Assuming that a roadway section of length L is to be drained, the total number of single inlets is:

$$N_s = \frac{L}{L_s} \qquad (12)$$

where N_s is the number of single inlets and L_s is the constant spacing of single inlets.

For twin inlets, the number of inlets N_t can be expressed as:

$$N_t = \frac{L}{L_{tw}} = \frac{L}{K_d L_s} \qquad (13)$$

where $K_d = Q_{int}/Q_{ins} > 1$ represents the ratio of the twin and single inlet capacities, and L_{tw} is the spacing of twin inlets.

Assuming that a single inlet installation costs C_s dollars and the twin installation costs $K_c C_s$ ($K_c > 1.0$), the economic feasibility of double inlets can be expressed as follows:

- Drainage Costs for Twin Inlets < Drainage Costs for Single Inlets

  or $$N_t \times K_c C_s < N_s \times C_s \qquad (14)$$

  After substitutions and simplifications, one obtains:

  $$K_c < K_d \qquad (15)$$

In other words, twin inlets are economically feasible only under those conditions where the relative increase in the inlet capacity due to the twin installation exceeds the relative increase in the inlet cost due to the twin installation.

Inlet costs were estimated here as follows:

- Single inlet structure —$1500
- Twin inlet structure —$1950
- Relative cost increase—K_c = 1950/1500 = 1.30

Note that the above calculation does not consider possible savings on the laterals connecting the inlet to the storm drain.

It would appear that twin inlets on a grade are not economically attractive unless the capacity increase due to the twin installation is at least 30%.

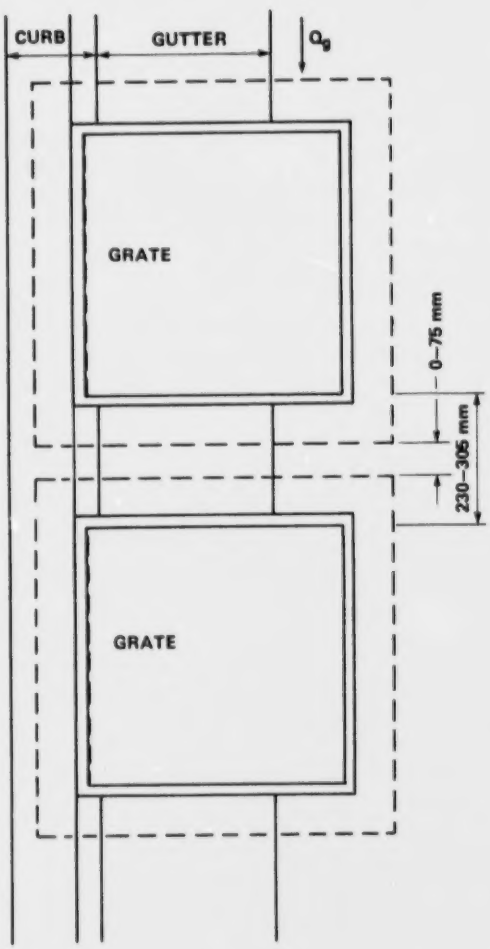


Figure 16/ Twin Inlet Installation

Twin Grate DD-713-A

The results of testing the twin-inlet DD-713-A are shown in Figure 17 and listed in the Appendix. The main objective of these tests was to determine the relative increase in the single inlet capacity achieved by adding the second grate in the twin installation. For each gutter flow, discharges were measured through the twin inlet and, immediately afterwards, through the single inlet which was obtained by blocking the downstream grate. These observations were made for grades varying from 0.003 m/m to 0.06 m/m, the crossfalls varying from 0.02 m/m to 0.06 m/m, and the spreads varying from 0.5 m to 3.0 m. The results were expressed as the ratio of the twin inlet capacity to single inlet capacity. Where necessary, interpolations and extrapolations of data were used to obtain information for the entire range of conditions.

On the average, the capacity of the twin-inlet DD-713-A was about 17% larger than that of the single inlet (i.e., $Q_{twin}/Q_{single} = 1.17$, standard deviation = 0.12, N = 144). For small spreads and low crossfalls, the twin inlet capacity exceeded only slightly the single inlet capacity. For largest spreads, the extrapolated data indicate that the twin inlet capacity would exceed that of the single inlet by 40% to 50%.

Twin Grate DD-705

The results of testing twin inlets with grate DD-705 are shown in Figure 18 and listed in the Appendix. The experimental procedure and scope of experiments were identical to those described earlier for twin inlets DD-713-A.

On the average, the capacity of the twin-inlet DD-705 was about 20% larger than that of the single inlet. The standard deviation about the mean for the ratio $R = Q_{in\ twin}/Q_{in\ single}$ was 0.18. For small spreads and low crossfalls, the capacity of the twin inlet exceeded only slightly the capacity of the single inlet. With increasing pavement spreads and longitudinal slopes, the deviations between the twin and single capacities also increased. For pavement spreads between 2.0 m and 2.5 m, the capacity of the twin inlet was about 30% to 40% higher than that of the single inlet.

The results obtained for twin-inlets DD-713-A and DD-705 are quite similar. Consequently, it was found unnecessary to test grate DD-713-B in a twin installation. It was recommended to adopt the capacity curves of the twin-inlet DD-713-A for the twin-inlet DD-713-B.

Considering the experimental findings presented above, one may draw the following conclusions for twin inlets.

1/ Capacities of the twin inlets tested (DD-713-A and DD-705) exceeded, on the average, the capacities of the corresponding single inlets by about 20%. The increase in capacity typically varies from few percent at low spreads and grades to 40% at large spreads.

2/ Cost estimates indicate that twin inlets may be about 30% more expensive than the corresponding single inlets (i.e., without considering possible savings on lateral connecting pipes).

Consequently, the twin inlets become economically feasible only if the increase in inlet capacity obtained by expanding a single inlet to a twin inlet is about the same. Under typical conditions (characterized by grades lower than 0.04 m/m, spreads less than 2.5 m, and the crossfall of 0.02 m/m), capacity increases around 30% will not be achieved. Twin inlets, therefore, offer few advantages in installations on a grade.

3/ The use of twin inlets should be primarily restricted to those conditions where the probability of inlet blockage and the resulting flood damage is high. Such conditions are frequently encountered in sags. For sag conditions, the twin inlet capacity exceeds the single inlet capacity by about 38% for small depths (say, $d < 0.2$ m; the weir mode of operation) and for large depths (say, $d > 0.3$ m) the twin inlet capacity would be about twice the single inlet capacity. For intermediate depths, there is a zone of transition from the 38% increase to the 100% increase (see Section 4.4).

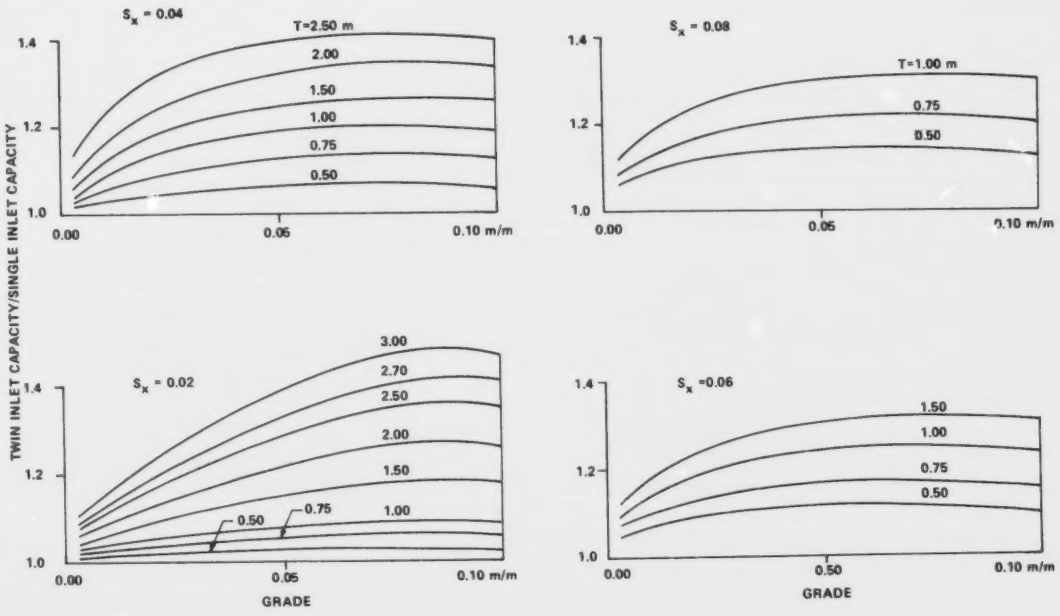


Figure 17/ Relationship Between Twin and Single Inlet Capacities: Curb & Gutter B, Inlet Grate Type DD-713-A

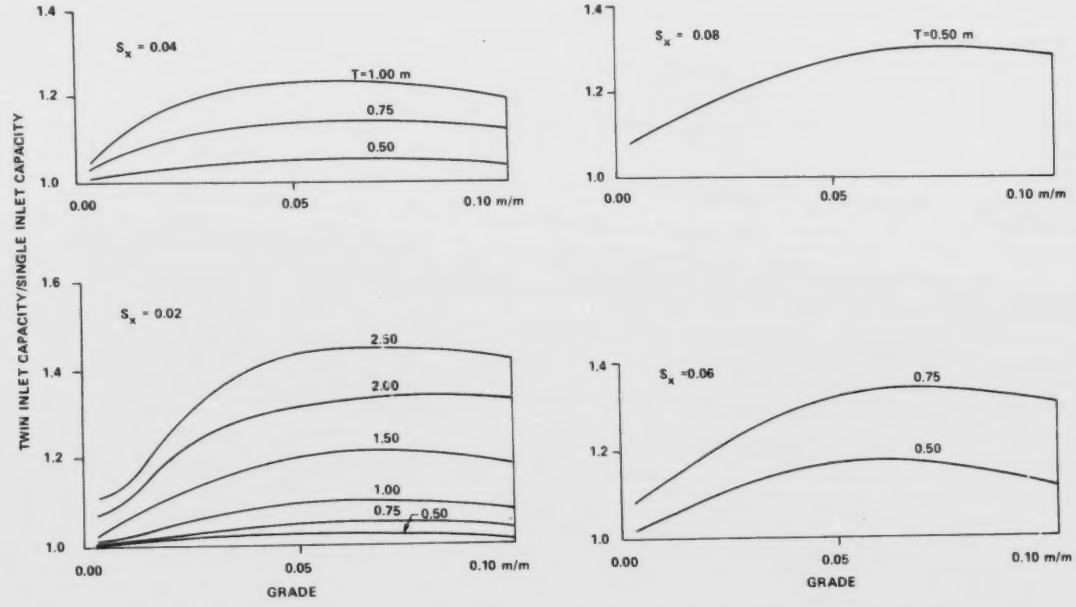


Figure 18/ Relationship Between Twin and Single Inlet Capacities: Curb & Gutter D, Inlet Grate DD-705

4.3.2.6/ Kitchener-Waterloo-Cambridge (KWC) Grate, Curb and Gutter B

The KWC grate (see Figure 19) was selected for testing because of its popularity in municipal design. A preliminary examination of the grate revealed that it has a fairly high area of openings (0.1680 m^2) and it is bicycle-proof. On the other hand, the hydraulically efficient slot at the curb, used in many MTC grates, is missing.

The KWC grate was tested for grades varying from 0.003 m/m to 0.06 m/m, crossfalls varying from 0.02 m/m to 0.06 m/m, and spreads varying from 0.5 m to 3.0 m. The results of these tests were then extrapolated for other grates, crossfalls and spreads, and all the data are presented in Figure 20 and listed in the Appendix.

Compared to other MTC designs (series DD-713 and DD-705), on a grade, the KWC grate yielded capacities about 20% lower. For low grades, the differences between the KWC grate and other MTC grates were almost negligible (less than 10%). These differences gradually increased with increasing grade. Extrapolated data indicate that for the extreme grade of 0.10 m/m, the KWC grate capacity would amount to about 77% of the other MTC grates named earlier.

In summary, on low grades (S > 0.03 mm), the KWC grate offers similar capacities to MTC grates DD-713-A and DD-713-B. For steeper grades (S < 0.03 m/m), the KWC grate has smaller capacities than the aforementioned MTC grates. As discussed later, the KWC grate can be used advantageously in a sag where it has the highest capacity of all the grates tested.

4.3.2.7/ Round Frame (RF) Grate, Curb and Gutter B

Poor fit of grates to frames leads to a rocking motion of the grate caused by vehicles passing over the grate. The resulting material fatigue may eventually lead to the structural failure of the grate. Such operational problems can be avoided if the grate base and the supporting part of the frame are machined. To reduce the cost of machining, circular grates and frame seats were proposed and one such design (Figure 21) was provided by MTC for the testing described here.

The RF grate tested is a bicyle-proof design with a total area of openings of about 0.1365 m^2. Compared to MTC grates DD-713-A and B, the RF grate has a slightly smaller porosity (by 6%) and lacks the hydraulically efficient slot at the curb.

The RF grate was installed with the curb and gutter B and subjected to limited testing for two grades (0.003 m/m and 0.02 m/m), one crossfall (0.02 m/m), and a full range of spreads.

In terms of capacities, the RF grate performed fairly well for low grades where the grade is a limiting factor for inlet capacity. On a grade of 0.003 m/m, the RF produced capacities amounting to about 90% of those observed for the DD-713 grates. On a grade of 0.02 m/m, the RF grate capacity started to fall off, amounting to about 80% of the capacities reported for standard MTC grates. One could speculate that for even steeper grades, the RF grate will lag further behind the standard designs.

For installations in a sag, the RF grate should yield capacities only slightly lower than those reported for grates DD-713-A and DD-713-B. This stems from similar effective perimeters and areas of openings.

In summary, on low grades (< 0.01 m/m) and in a sag, the RF grate offers hydraulic capacities similar to those reported for other MTC grates. For higher grades (S > 0.01 m/m), the RF grate capacity falls significantly below that reported for common MTC grates.

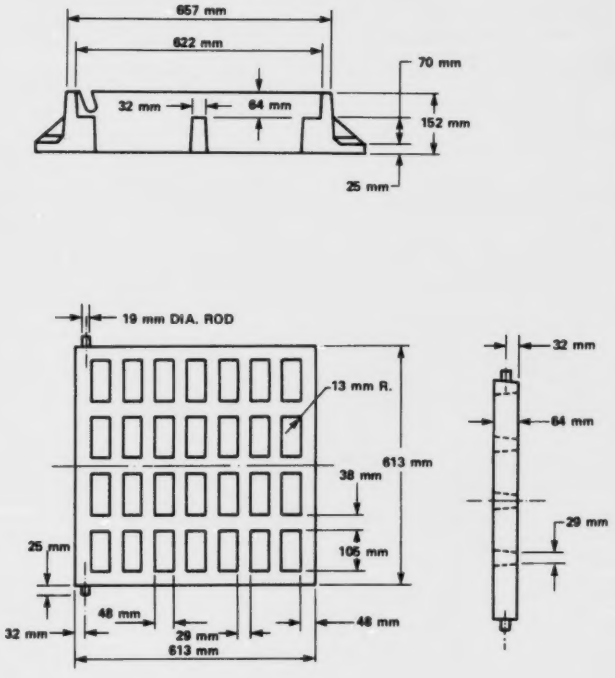


Figure 19/ Kitchener-Waterloo-Cambridge (KWC) Inlet

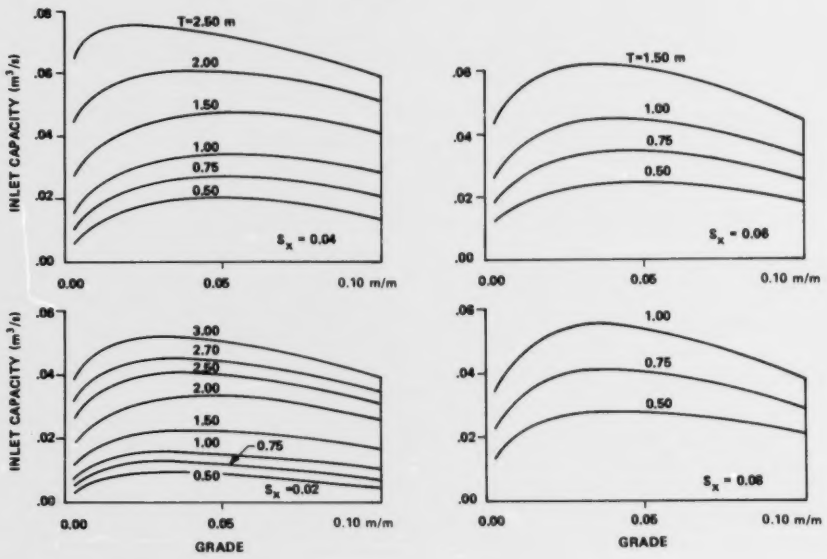


Figure 20/ Sewer Inlet Capacity: Curb and Gutter Type B, KWC Inlet Grate

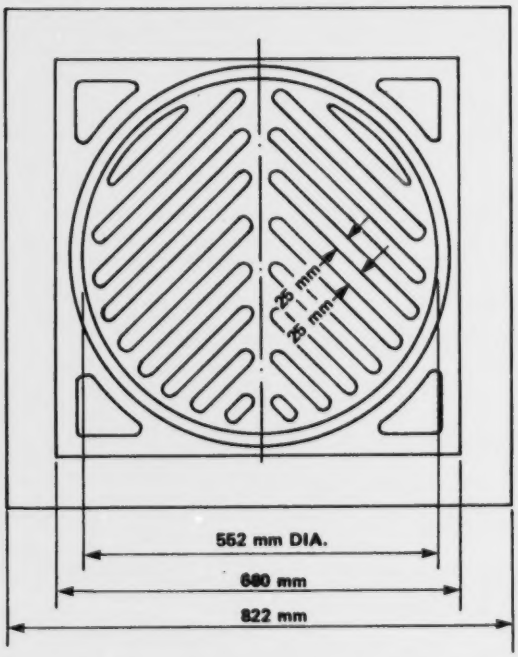


Figure 21/ Round Frame Grate

4.3.2.8/ New Grate Designs

Under this task, capacities of a number of new grate designs were studied. The primary purpose was to compare various designs and to identify grate features which were likely to increase grate capacity. Altogether, 10 new grate designs (see Figure 22) were proposed, manufactured of wood, tested in a rather limited experimental programme, and evaluated. A description of individual designs follows.

Design No. 1

The outside grate dimensions for all the new designs were identical to those of grates DD-713-A and DD-713-B. Design No. 1 has standard diagonal bars of 25 mm thickness and an identical width of slots. The angle between the bars and the curb is 30°. There is no supporting crossbar.

Design No. 2

Design No. 2 is similar to Design No. 1 except that the angle between the bars and the curb was changed from 30° to 45°.

Design No. 3

Design No. 3 is similar to Design No. 2 except for a partly depressed crossbar. The crossbar is depressed 25 mm below the grate surface.

Design No. 4

Design No. 4 is similar to Design No. 2 except for a crossbar flush with the grate surface.

Design No. 5

Design No. 5 has standard bars parallel to the curb. The bar width and the slot width are 25 mm. There is no supporting crossbar.

Design No. 6

Design No. 6 is similar to No. 5 except for a partly depressed crossbar located 25 mm below the grate surface.

Design No. 7

Design No. 7 is similar to No. 5 except for a flush crossbar.

Design No. 8

Design No. 8 is similar to No. 5 except for the cross-section of the bars. The upper bar edges were rounded off (r = 6 mm) to improve the hydraulic efficiency of this grate.

Design No. 9

Design No. 9 is similar to No. 8 except for the spacing of bars, which was increased from 50 mm (centre-to-centre) to 64 mm.

Design No. 10

Design No. 10 is similar to No. 8 except for the spacing of bars, which was increased from 50 mm (centre-to-centre) to 75 mm.

In summary, the above 10 designs include both diagonal and parallel-to-curb bar grates. For diagonal grates, the bar angle and the crossbar arrangement were varied. For parallel bars, the bar cross-section and spacing, and the crossbar arrangement were varied.

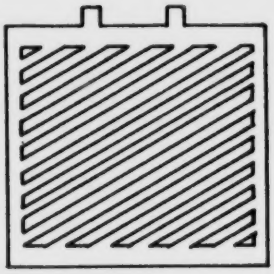
The testing of new designs was limited to two grades (0.003 m/m and 0.02 m/m), one crossfall (0.02 m/m) and a full range of spreads (0.5 m to 3.0 m).

The results of these screening tests were evaluated on a comparative basis. For each grate, a mean capacity was determined by integrating the capacity vs. spread curve from T = 0.0 m to T = 3.0 m, and dividing the result by the total spread T = 3.0 m. For each grade, the mean values were grouped together and presented as fractions of the data set mean in Table 3.

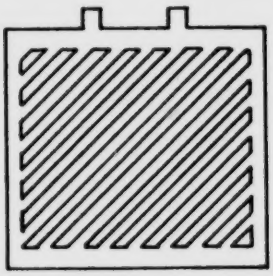
It is obvious from Table 3 that grate capacities were affected surprisingly little by the grate geometry. For both grades, the variation about the mean is about ± 8%. Such a variation is even less than the required accuracy of inlet capacity curves of ± 10%. Under these circumstances, one should not attempt to evaluate the individual grate designs on the basis of relative capacities given in table 3. The variations in these capacities are not statistically significant and may be caused by the measurement and interpolation errors, rather than by the actual performance of various designs. It can be concluded that for low grades the inlet capacity is primarily controlled by the gutter discharge and the arrangement of grate bars has very little effect on the grate capacity. (Note that a similar finding for inlets in a sag and low depths of ponding is discussed in Section 4.4.2.)

Table 3/Relative Capacities of New Designs

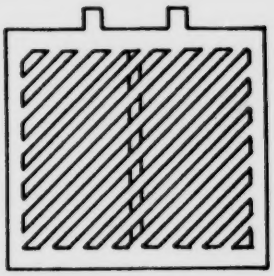
		Design Number									
Grade		1	2	3	4	5	6	7	8	9	10
0.003 m/m	Grate Capacity / Group Mean Capacity	1.08	1.06	1.01	1.06	0.95	0.97	0.97	0.94	0.93	1.03
0.02 m/m	Grate Capacity / Group Mean Capacity	1.00	0.91	0.99	0.96	1.07	1.06	1.01	1.00	0.97	1.03
Approximate Area of Openings (m^2)		0.16	0.16	0.16	0.15	0.16	0.16	0.15	0.16	0.19	0.20



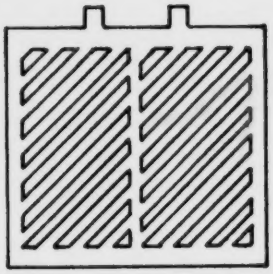
DESIGN No 1
DIAGONAL BARS (30°)



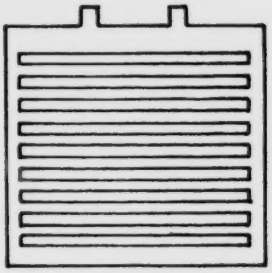
DESIGN No. 2
DIAGONAL BARS (45°)



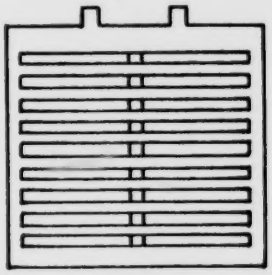
DESIGN No. 3
DIAGONAL BARS,DE-
PRESSED CROSSBAR



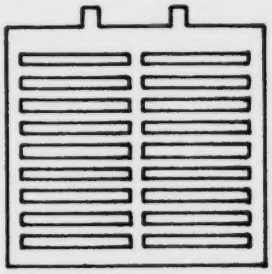
DESIGN No. 4
DIAGONAL BARS,
FLUSH CROSSBAR



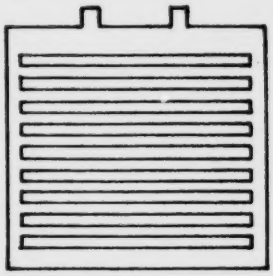
DESIGN No. 5
PARALLEL BARS,
SPACING 50 mm



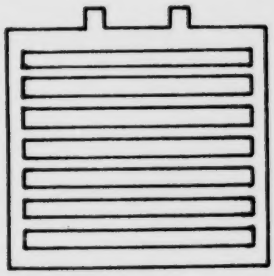
DESIGN No. 6
PARALLEL BARS,
DEPRESSED CROSSBAR



DESIGN No. 7
PARALLEL BARS,
FLUSH CROSSBAR

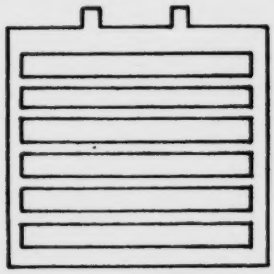


DESIGN No. 8
ROUNDED-OFF
PARALLEL BARS,
SPACING 50 mm



DESIGN No. 9
ROUNDED-OFF
PARALLEL BARS,
SPACING 64 mm

NOTE:
GRATE BARS USED IN
DESIGNS Nos. 1-7 HAVE
SQUARE EDGES. THE
BAR WIDTH IS 25 mm,
THE BAR SPACING IS
50 mm (FROM CENTRE
TO CENTRE)



DESIGN No. 10
ROUNDED-OFF
PARALLEL BARS,
SPACING 75 mm

Figure 22/ New Grate Designs Tested

Another explanation of the observed low sensitivity of grate capacities to the arrangement of bars can be obtained by looking at the hydraulic efficiencies of standard grates DD-713-A and DD-713-B. Such hydraulic efficiences are defined here as the grate discharge divided by the calculated gutter discharge corresponding to the grate width (i.e., measured from the curb). Note that this definition differs from the conventional one in which the total gutter flow, over the full flow spread, is considered. The definition proposed here is justified by an argument that only water particles passing over or close to the inlet opening can be intercepted by the inlet. Indeed, a water particle travelling, say, 3 m from the curb cannot be intercepted even by a completely open inlet without any grate.

Following the above procedure, the efficiency of the DD-713-A grate (crossfall = 0.02 m/m, curb and gutter B) was defined as follows:

Grade (m/m)	0.003	0.01	0.02	0.03	0.04	0.06	0.08	0.10
Efficiency (%)	95	81	65	57	51	42	36	31

Neglecting the lateral inflow (this seems to be acceptable for a small crossfall of 0.02 m/m), an open inlet without any grate would have efficiencies close to 100%. For low grades, the efficiencies of the existing inlets are fairly high and significant improvements cannot be achieved by changes in the bar arrangement. New grates with improved capacity could be developed only for steep grades where as little as 31% of water passing over the grate is intercepted. It appears unlikely that further studies of the inlet bar arrangement would yield significant improvements in inlet capacity. Other approaches, such as increases in the inlet dimensions and increased concentration of the gutter flow over the inlet, may offer better solutions.

Several weak tendencies observed in the tests of new grate designs are described below.

The diagonal grates performed better than the parallel bar grates for the low grade used (0.003 m/m). It would appear that diagonal bars deflect the flow towards the curb and thereby increase the inlet capacity. The best results were obtained for the 30° diagonal grate without the crossbar. For parallel bars on the low grade, capacities were improved by removing the crossbar and increasing the spacing from 50 mm to 75 mm.

On the steeper grade (0.02 m/m), the parallel bar grates performed better than the diagonal bars. It is conceivable that the increased gutter flow velocity interferes more with the inlet operation for diagonal bars than the bars parallel to the curb. A flush crossbar reduced the inlet capacity. There was no noticeable effect of the rounding-off of bar edges. Inlet capacities were again improved by increasing the bar spacing from 50 mm to 75 mm.

In summary, the observed capacities of new inlets on low grades were little sensitive to the arrangement of grate bars. On steeper grades ($\geqslant$ 0.02 m/m), the grates with bars parallel to the curb seem to perform better than the diagonal bar grates. Improved capacities were obtained by removing or depressing the crossbar and by significantly increasing the bar spacing. For the grates tested, the newly proposed grates did not offer any significant improvement compared to the common MTC grates.

4.4/ Grate Inlets in a Sag

4.4.1/ Introduction

The hydraulics of grate inlets in a sag are described in Reference 4. A brief summary of that description follows:

A grate inlet in a sag operates first as a weir having a crest length roughly equal to the outside perimeter P along which the flow enters. Bars are disregarded and the side against the curb is not included in computing P. Thus, the perimeter P equals the sum of the grate length and twice the grate width. Weir operation continues to a depth of about 0.12 m above the top of grate and the discharge intercepted by the grate is:

$$Q_i = 1.6535\ P\ d^{1.5} \qquad (16)$$

where Q_i = rate of discharge into the grate opening (m^3/s)
P = perimeter of grate opening (m), disregarding bars and neglecting the side against the curb
d = depth of water at grate (m)

When the depth at the grate exceeds about 0.43 m, the grate begins to operate as an orifice and the discharge intercepted by the grate is:

$$Q_i = 2.96\ A\ d^{0.5} \qquad (17)$$

where Q_i = rate of discharge into the grate opening (m^3/s)
A = clear opening of the grate (m^2)
d = depth of ponded water above the top of grate (m).

According to Reference 4, between depths over the grate of about 0.12 m and 0.43 m, the operation of the grate inlet is indefinite due to vortices and other disturbances, and the capacity of the grate is somewhere between those given by Equations 16 and 17.

Inlets in sags are susceptible to clogging, which may substantially reduce their capacity and result in serious flood damages. It is therefore recommended [4] to consider the capacities of inlets in sags reduced, by clogging, to 50% of the values calculated from Equations 16 and 17 (i.e., using a design safety factor of two). Also for design safety, three inlets are installed in a sag vertical curve as described in Section 8.6.1.

In view of the lack of experimental data on operation of grate inlets in sags, further investigations of inlets in sags were undertaken in this study. The main objectives of these investigations were to produce capacity curves for selected inlets in a sag and to verify the earlier described computational procedure.

The inlets were tested in the existing experimental facility which was described earlier. The testing programme is summarized in Table 4.

4.4.2/ Inlet Capacities Observed for Grates DD-713-A, DD-713-B, DD-705, Twin DD-713-A, and KWC

The experimental programme listed in Table 4 was designed to indicate if capacities of inlets in a sag were affected by the approach grade and by the crossfall.

The effects of the approach road grade on the capacity of inlets in a sag were evaluated for grate DD-713-A. The grade was varied in three steps, from 0.003 m/m to 0.02 m/m. Experimental data observed for various grades were plotted in Figure 23. It can be inferred from Figure 23, that the observed data do not indicate any clear dependence of the capacities on the approach grade. For larger depths (say, $d > 0.15$ m), there may be some weak indications that higher capacities were observed for higher grades. Such effects are insignificant particularly if one considers the overriding accuracy criterion of $\pm$ 10%. Consequently, it was concluded that the capacity of inlets in a sag is not significantly affected by the approach grade (at least not within the range tested) and the testing programme for other inlets could be reduced to one grade. Note also that, in real life, the approach to the sag will be characterized by a gradual reduction in the grade and that grades steeper than 0.02 m/m will not occur close to the sag.

The effects of crossfall on the capacity of inlets in a sag were evaluated for all the inlets tested. For that purpose, 21 inlet capacity curves were divided into three groups, according to the crossfall, and each capacity curve was characterized by a mean discharge for the range of depth from 0.00 m to 0.30 m. Mean discharges for each of the three groups were then compared to the mean of all 21 sets and the following results were obtained:

• S_x(m/m)	0.02	0.04	0.06
• Group Mean/Total Mean	0.960	1.006	1.034

While the above-described data indicate a weak dependence of mean capacities on the crossfall, the observed variations are negligible and were neglected in further analysis for the sake of simplicity.

The experimental capacity curves for inlets DD-713-A, DD-713-B, and DD-705 are plotted in Figure 24. The capacity curves in Figure 24 are nearly identical. A comparison of the opening areas for the three grates under consideration yielded the following results:

• Grate	DD-713-A	DD-713-B	DD-705
• Openings area (m^2)	0.1455	0.1353	0.1432

It would appear that the capacity of an inlet in a sag is not affected by the arrangement of inlet bars, but may be affected by the grate perimeter and the total area of openings, as discussed later. Consequently, for the above three inlets with similar areas of openings, a single capacity curve was adopted. This capacity curve is plotted in Figure 24 and the numerical values are listed in the Appendix.

Table 4/ Grate Inlets In A Sag, Testing Programme

Curb & Gutter Type	Inlet Grate Type	Grade (m/m)	Crossfall (m/m)	Depth (m)
B	DD-713-A	0.003, 0.01, 0.02	0.02, 0.04, 0.06	0.03—0.28
B	Twin DD-713-A	0.01	0.02, 0.04, 0.06	0.02—0.18
B	DD-713-B	0.01	0.02, 0.04, 0.06	0.04—0.27
B	KWC	0.01, 0.02	0.02, 0.04, 0.06	0.03—0.24
D	D-705	0.01	0.02, 0.04, 0.06	0.03—0.26

The next grate tested was the KWC grate. It was noted that, although this grate had the same perimeter as the three grates tested previously, it had a larger area of openings (A = 0.1680 m^2). Consequently, a separate capacity curve was developed for the KWC grate and presented in Figure 25. Capacities of the KWC inlet for various depths of ponding are also listed in the Appendix. A comparison of the KWC curve to that developed for the three MTC grates indicated that for low depths both curves were practically identical. This can be explained by the fact that all the grates discussed have an identical effective perimeter which controls the inlet capacity. In the transition zone from weir operation to orifice operation (d > 0.15 m), the large area of openings of the KWC grate starts to control the inlet capacity and, consequently, the KWC grate yielded increasingly higher capacities compared to those observed for other MTC grates characterized by smaller areas of openings (A = 0.145 m^2). At even higher depths, the grates operate as an orifice and, consequently, the KWC grate has larger capacities than the others because of its larger area of openings.

The last inlet tested was a twin-inlet DD-713-A. The experimental curve found for this inlet is shown in Figure 26, together with a curve for a single-inlet DD-713-A. It can be inferred from Figure 26 that the twin inlet has larger capacities than the single inlet. For low depths, this increase in capacities is about 35% and becomes even larger for increasing depths. The limitations of the testing apparatus did not allow testing of the twin DD-713-A for flows larger than about 0.2 m^3/s (h = 0.16 m). It can be only speculated that when the inlet operates in the orifice mode (h > 0.25 m), the capacities of the twin inlet would be about twice the capacity of the single inlet. The capacity curve of the twin inlet DD-713-A was extrapolated for depths larger than 0.16 m and listed in the Appendix. The extrapolation was based on an assumption that the twin inlet capacity in excess of the single inlet capacity would linearly increase from 42%, observed for depth 0.16 m, to 100% at depth 0.30 m.

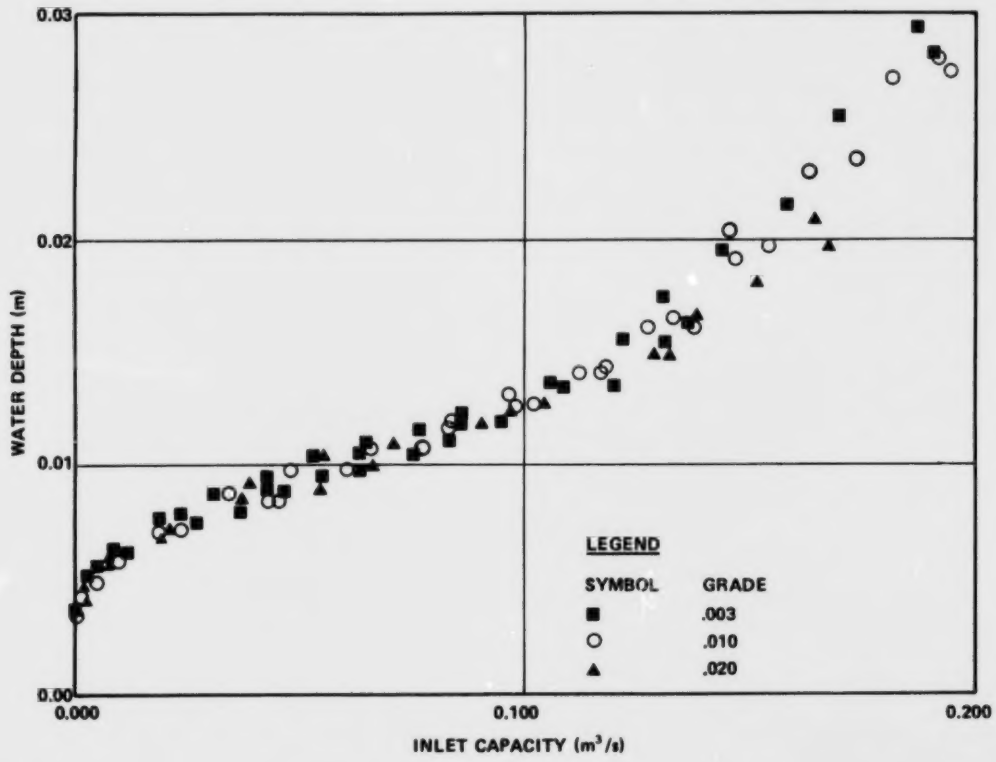


Figure 23/ Capacity of Inlet DD-713-A in a Sag for Various Approach Grades

4.4.3/ Inlet Operation Mode

The second objective of the study of inlets in a sag was to verify the applicability of the existing computational procedures (Section 4.4.1) to inlet design. In those procedures, it is assumed that the inlet operates first as a weir (the presence of grate bars is neglected) and for higher depths, as an orifice.

The verification results are presented in Figure 27. Curves 1 and 2 represent the inlet capacity curves for the three grouped inlets (DD-713-A, DD-713-B, and DD-705) and for the KWC grate, respectively.

Curves 3 and 4 represent capacities calculated from the weir equation (Equation 16) presented earlier. The perimeter was taken as P = 1.3886 m. Curve 3 was determined for weir heads taken as the maximum depth of water in the gutter. Curve 4 was calculated for an opening area-weighted mean depth of water over the grate. Note that grates are typically installed on a cross-fall of 0.125 m/m and the depth over the grate will therefore vary.

Curves 5, 6, and 7 were calculated from the orifice equation. The first two, 5 and 6, were calculated for the group of three inlets characterized by the opening area of 0.145 m^2. When computing Curve 5, Equation 17 was used and the head was defined as the maximum depth in the gutter. For Curve 6, a mean head over the grate was used and the orifice coefficient of discharge was taken as 0.623 (after Reference 2) compared to 0.67 used in Equation 17. Curve 7 is analogous to Curve 6, except it corresponds to the opening area of 0.168 m^2 which was measured for the KWC grate.

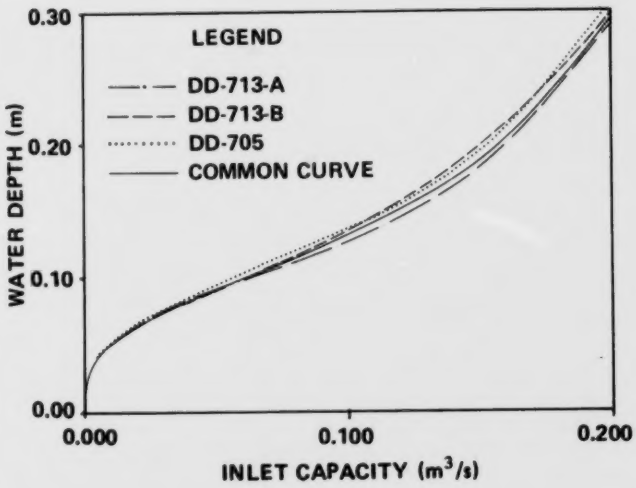


Figure 24/ Capacity Curves for Inlets DD-713-A, DD-713-B, and DD-705 in a Sag

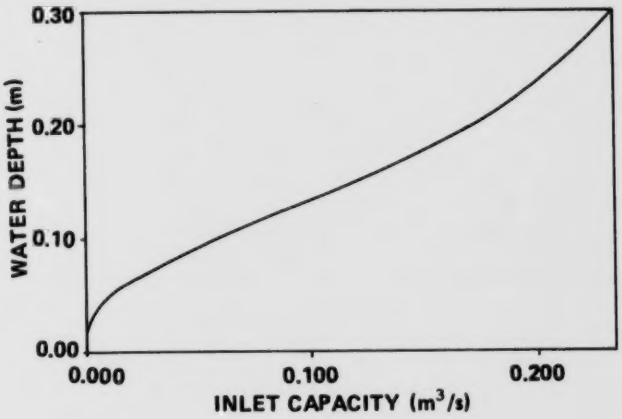


Figure 25/ Capacity Curve for KWC Inlet in a Sag

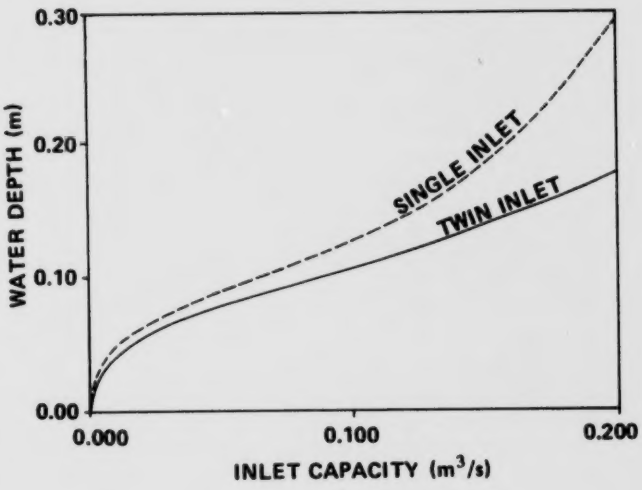


Figure 26/ Capacity Curve for Twin Inlet DD-713-A in a Sag

When comparing the experimental and calculated capacity curves in Figure 27, it appears that the inlet capacities can be described by the weir and orifice equations. For low heads, the experimental Curves 1 and 2 are nearly identical and can be well approximated by the weir Curve 4. The agreement between the experimental curves, for low heads, follows from the fact that all the inlets discussed have the same perimeter. For higher heads, the curves seem to enter the orifice region where the capacities depend on the area of openings and, consequently, the KWC inlet with a larger area yields higher capacities. The correction of weir heads for variation over the grate width was implemented in Curve 4 and yielded better results than the computation based on the maximum depth only (Curve 3).

LEGEND

1 - EXPERIMENTAL CURVE FOR INLETS DD-713-A, DD-713-B, and DD-705
2- EXPERIMENTAL CURVE FOR KWC INLET
3- WEIR EQUATION (DEPTH MEASURED AT THE CURB)
4- WEIR EQUATION (A WEIGHTED-MEAN DEPTH USED)
5 - ORIGINAL ORIFICE EQUATION FOR INLETS DD-713-A, DD-713-B, AND DD-705 (DEPTH MEASURED AT THE CURB)
6 - MODIFIED ORIFICE EQUATION FOR INLETS DD-713-A, DD-713-B, AND DD-705 (MEAN DEPTH USED)
7 - MODIFIED ORIFICE EQUATION FOR KWC INLET (MEAN DEPTH USED)

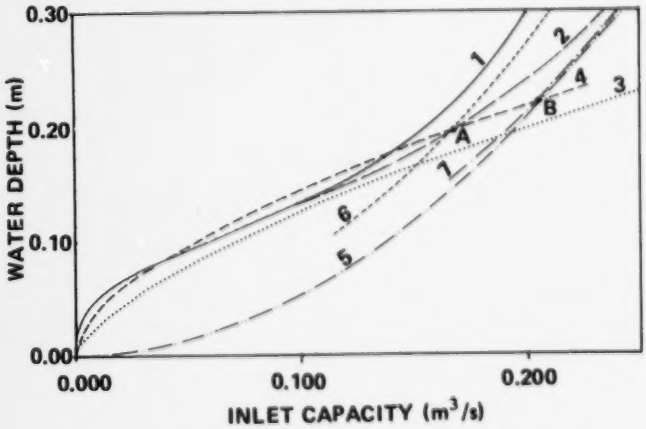


Figure 27/ Experimental and Calculated Capacity Curves for Various Inlets in a Sag

For depths of flow larger than about 0.20 to 0.22 m, the experimental curves are better approximated by the orifice equation. Consideration of a mean head over the grate and of a more realistic discharge coefficient (Curves 6 and 7) yielded better results than the earlier-presented orifice equation applied for the maximum depths in the gutter (Curve 5).

An earlier discussion of the twin DD-713-A inlet indicated that the twin inlet capacity was about 35% larger than that of the single inlet. The effective perimeter of the twin inlet would be about 1.92 m, neglecting the sides next to the curb and the joint side in the centre. Compared to the single inlet, the twin inlet effective perimeter is about 38% larger and this increase confirms the observed increase in capacities. Because of experimental limitations, the tests could not be extended to the orifice mode of operation.

When comparing capacities of sag and grade installations for identical spreads, inlets in a sag were found to have higher capacities in general than those on a grade. This was particularly true for higher depths of ponding. For very low depths of ponding (say, less than 0.07 m), inlets in a sag have rather low capacities which may be even lower than those of inlets on a grade. This was particularly observed in installations without gutters.

Considering the data presented in this section, one can conclude that the experimental capacities of inlets in a sag can be approximated by a weir equation in the region of low heads (say, $d < 0.2$ to 0.22 m) and by an orifice equation for large heads ($d > 0.2$ to 0.22 m). Compared to the earlier results, the following new findings were made:

- The head in both the weir and orifice equation should be taken as a mean head over the grate.
- The orifice discharge coefficient of 0.623 (a mean of values given in Reference 2) seems to be a better choice, for the inlets tested, than the value of 0.67 built into the design formula (Equation 17).
- The observed inlet capacities were approximated fairly well by the weir equations for depths up to 0.20 m.

4.5/ Combination Inlet C93 on a Grade

Combination inlets provide both a curb opening (sometimes protected by a grate) and a grate opening. Such inlets may offer advantages of both types of openings. Common combination inlets have the curb opening directly opposite the grate and their use is particularly desirable in a sag. The curb opening provides a relief opening should the grate become clogged.

Another type of combination inlet is the so-called *sweeper* inlet which has a curb opening immediately upstream of the grate. In this arrangement, the curb opening functions as a trash interceptor during the initial phases of a runoff event. In a sag, the sweeper inlet may have curb openings on both sides of the grate.

Investigations of combination inlets in this study were limited to the testing of one inlet selected by MTC. Apart from the total capacity of the inlet, the effectiveness of the curb opening was also of interest.

The combination inlet selected for testing was model C93, shown in Figure 28. The grate of this model is a bicycle-safe type with a bar arrangement similar to inlet DD-713-B. The area of grate openings is about 0.113 m^2, appreciably smaller than that reported for standard MTC inlets in the DD-713 series (0.145 m^2). The reduced openings area was partly caused by the absence of a wide slot next to the curb as typical for MTC grates. The curb opening consists of four separate openings each 125 mm wide and 64 mm high. These curb openings are somewhat offset inside the curb. The total area of openings for the C93 inlet was about 0.145 m^2.

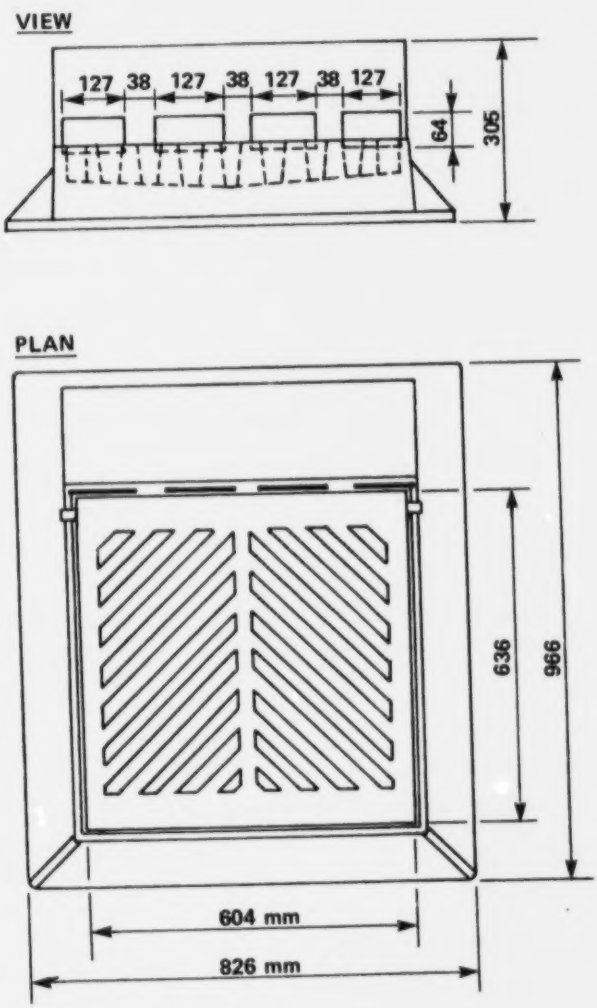


Figure 28/ Combination Inlet C-93

The combination inlet was tested for grades varying from 0.003 m/m to 0.06 m/m, crossfalls varying from 0.02 m/m to 0.06 m/m, and spreads varying from 0.0 m to 3.0 m. The experimental data were then extrapolated for other grades and crossfalls. The test results are shown in Figure 29 and listed in the Appendix.

The combination inlet behaved similarly to the grate inlets tested earlier. Observed capacities increased rapidly with increasing grade up to a certain point, beyond which the inlet capacity started to fall off. On low grades (0.02 m/m), the inlet capacity is clearly controlled by the gutter flow which limits the supply of water to the inlet. Once the gutter flow velocity starts to interfere with inlet operation, inlet capacities start to slowly diminish.

Observed inlet capacities increased with both increasing spread and increasing crossfall. Both these conditions lead to increased depths of flow over the grate and, therefore, to increased flow interception.

The contribution of the curb openings to the total intercepted flow was also of interest. These openings represent about 28% of the total openings area of the C93 inlet.

The flow interception by the curb openings was observed in each run by measuring the inlet discharge for two conditions—with curb openings open, and curb openings closed. Such observations provided data on the discharge through the curb openings. For the conditions studied, the curb openings contributed, on the average, only 5.7% (standard deviation 0.034) of the total inlet discharge. Such contributions were disappointingly low and indicated that for a non-clogged combination inlet, the contribution of the curb openings to the overall capacity is negligible. This follows from the fact that, close to the curb, the velocity vector is more or less parallel to the opening plane. Furthermore, since the openings are slightly offset inside the curb, flow separation occurs at the upstream joint between the curb and inlet and further reduces the inflow into the curb openings.

It is conceivable that the combination inlet would perform better in a sag installation, where the inlet capacity would be enhanced by the improved performance of the curb openings. Although these openings represent 28% of the grate opening area for the model tested, the capacity in a sag installation would not increase by the same percentage since the head over the curb openings would be appreciably smaller than that over the grate.

It was also of interest to compare the combination inlet C93 to the MTC grate DD-713-B which has a similar arrangement of bars. The comparison indicated that, on the average, grate DD-713-B intercepted about 30% more flow than grate C93. Such a difference in performance is significant, considering the accuracy of the presented data as $\pm$ 10%. An explanation for the difference in performance of these two grates was found by comparing the grate opening areas. Neglecting the curb openings (which contribute very little to the total interception), the opening area of inlet C93 (0.113 m^2) represents 0.78 of the area estimated for grate DD-713-B. This difference in areas is then caused by the fact that the

wide, hydraulically efficient slot used in grate DD-713-B is missing in inlet C93. The hydraulic efficiency of this slot follows from the slot location, next to the curb, where the largest flow depths are encountered, and from a hydraulically efficient entry to the slot—the flow is guided by the curb on one side and, on the other side, the slot edge is rounded. Estimates for sag conditions indicate that this slot may contribute as much as 30% of the total inlet discharge.

It can be concluded that for installations on a grade, the combination inlet tested does not offer any advantages over the other grate inlets tested. On the contrary, the combination inlet C93 has average capacities about 30% lower than those of similar MTC grate inlets tested. These lower capacities follow from the fact that the opening area of the C93 grate is only 0.113 m^2 as compared to 0.145 m^2 measured for common MTC inlet grates.

Combination inlets should be considered primarily in sag installations where the probability of grate blockage is increased. Curb openings are less susceptible to clogging than grate openings and can, therefore, provide relief if the grate is completely clogged. Disregarding inlet blockage, inlet C93 would not yield higher discharges in a sag installation than MTC grates DD-713-A and DD-713-B, since all three inlets have approximately the same total area of openings.

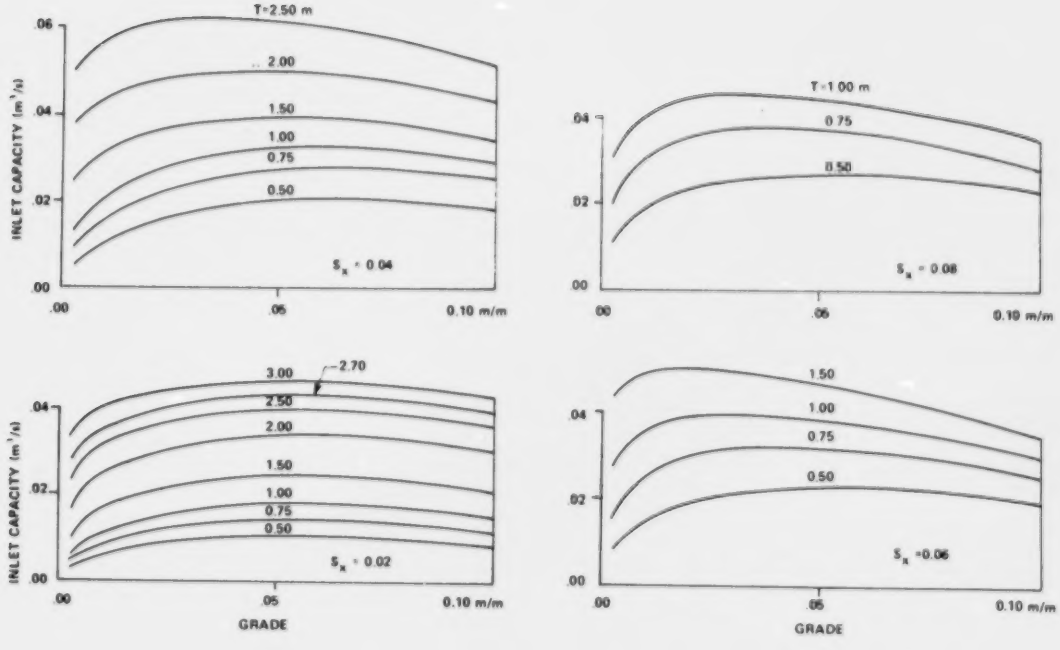


Figure 29/ Sewer Inlet Capacity: Combination Inlet C-93, Curb & Gutter B

5/ Ditch Inlets

The removal of water from medians and drainage ditches is accomplished by ditch inlets, which are generally drop structures covered by grates. Grates are typically installed flush with the face of an earth bank placed across the ditch (see Figure 30). Such installations protect the inlet structure and increase inlet capacity by ponding water over the inlet and eliminating carryover flow typical for other types of inlets.

The design of ditch inlets must consider two interrelated facets—the hydraulic capacity and maintenance problems. Both these facets were considered in the study reported here. The primary objective was to evaluate several existing and new designs and make recommendations regarding their applications. Towards this end, five grate designs were tested for slopes 1:1 and 2:1. Apart from the hydraulic testing, qualitative tests of inlet susceptibility to clogging were also undertaken. A detailed description follows.

5.1/ Ditch Inlets Tested

Altogether five inlet grates were tested (see Figure 31). Grate Design A is referred to as a *honeycomb grate* and corresponds to MTC grate DD-710. Designs B and C are characterized by parallel longitudinal bars arranged in vertical planes. Grate Design B has a 127 mm (5 inch) spacing of bars and Design C has a reduced spacing of 63.5 mm (2.5 inch). Grate Designs B and C were patterned after the Burlington inlet. The last two grates, Designs D and E, employ crossbars spaced 127 mm (5 inch) and 63.5 mm, respectively. All five grates were 0.39 m wide and 1.50 m long.

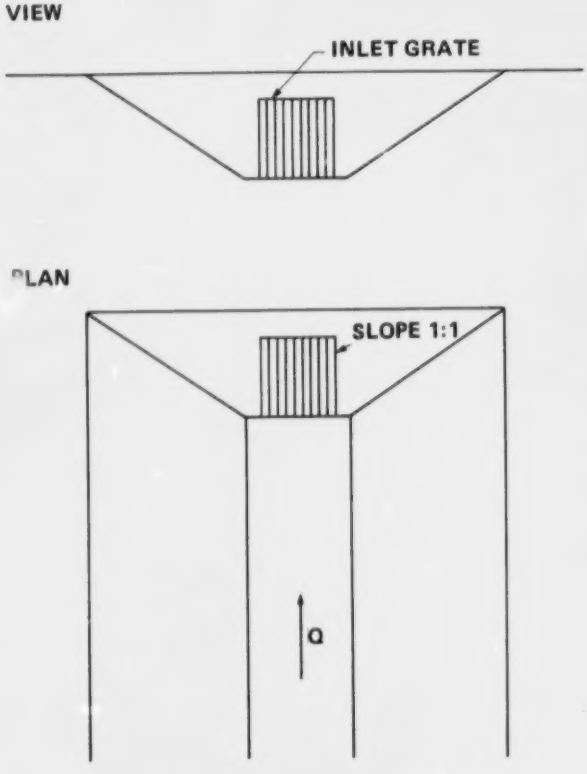


Figure 30/ Ditch Inlet

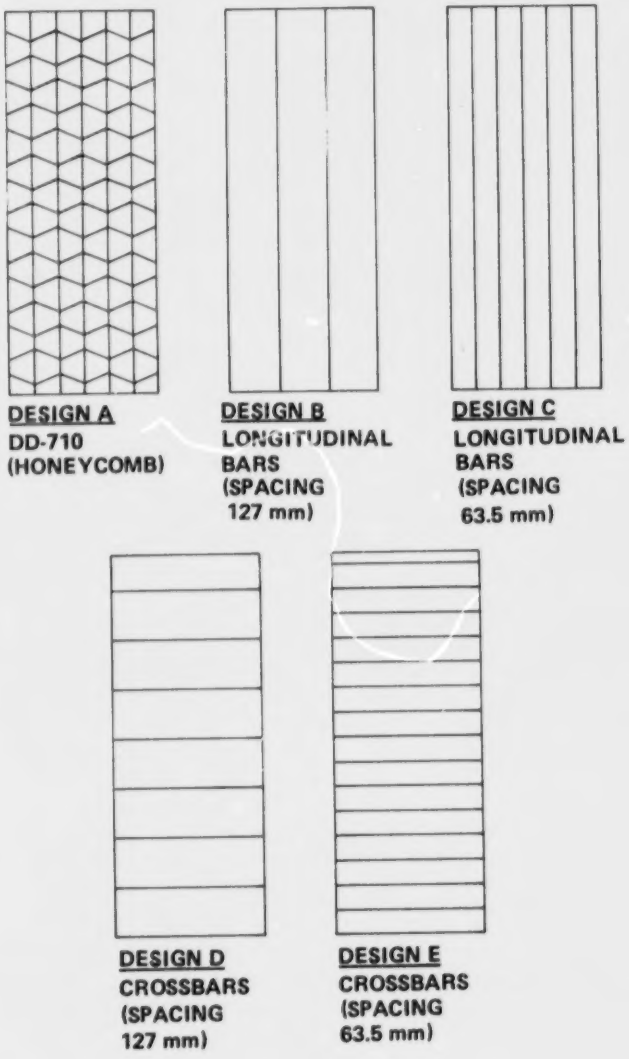


Figure 31/ Ditch Inlet Grates Tested

5.2/ Experimental Apparatus and Procedures

The testing of inlets was undertaken in the one-metre flume. At the tail end of the flume, an inlet structure was installed, eliminating the possibility of a grate submergence. The inlet structure consisted of a sloping plane, placed across the flume, with a frame which would accommodate individual grates. Two grate slopes were used —1:1 and 2:1.

Altogether, 10 inlet capacity curves were developed for the five designs tested and the two slopes. During the investigations of inlet clogging, two types of floating debris were used—straw, and plastic sheets (0.15 m x 0.15 m). Preliminary tests indicated that the entrapment of debris by the inlet grate was not affected by the discharge or the depth of flow. Consequently, all the comparative tests were done for a flow discharge of 0.1 m^3/s and a depth of 0.22 m. In tests with the straw, a known weight of dry straw was floated on the water surface well upstream of the inlet. Any straw trapped by the grate was collected, dried, and weighed to determine the percentage of straw trapped by each grate. In tests with plastic sheets, five sheets (0.15 x 0.15 m) were placed on the water surface well upstream of the grate and the number of sheets trapped was recorded. Experiments with both types of debris were repeated three times to verify the repeatability and consistency of results.

5.3/ Hydraulic Capacity of Ditch Inlets (DD-7, Modified Burlington Grates, and Crossbar Grates)

In a preliminary series of tests, the effects of the longitudinal slope of the ditch and of the width of the inlet on the observed capacities were evaluated. The results of these evaluations are shown in Figure 32 and indicate that, for the desired accuracy limits of $\pm$ 10%, the ditch grade can be neglected and the inlet width can be incorporated in the design curves as a linear parameter.

Experimental capacity curves for the five grates tested are shown in Figures 33 and 34 and listed in the Appendix. To obtain the discharge for a particular inlet, the designer has to multiply the capacity read from Figures 33 and 34 by the inlet width.

A brief examination of Figures 33 and 34 shows that all the grates tested are fairly efficient. Compared to a free opening, the five designs tested typically conveyed 85% to 90% of the free-opening capacity. Considering both grate slopes, the highest capacities were observed for Design B, followed by Designs A and D. The lowest capacities were observed for Designs C and E. Thus, the highest capacities were observed for widely-spaced longitudinal bars. The MTC honeycomb grate performed about as well as the widely-spaced crossbar grate. The lowest capacities were observed for longitudinal bar and crossbar grates with reduced spacing.

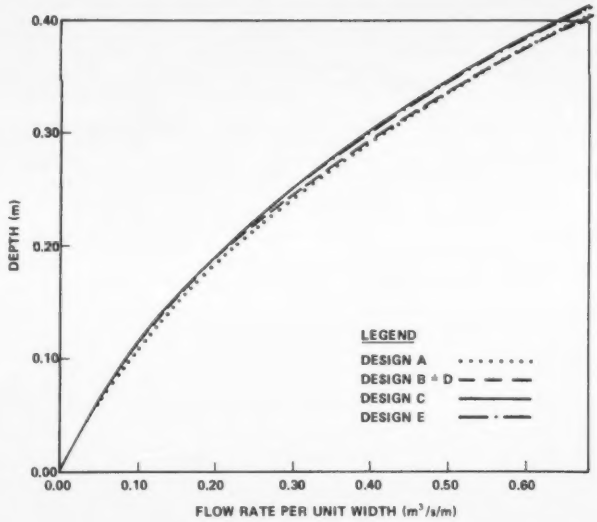


Figure 33/

Capacity Curves for Ditch Inlet Grates Tested, Grate Width = 1.0 m, Grate Slope 1:1

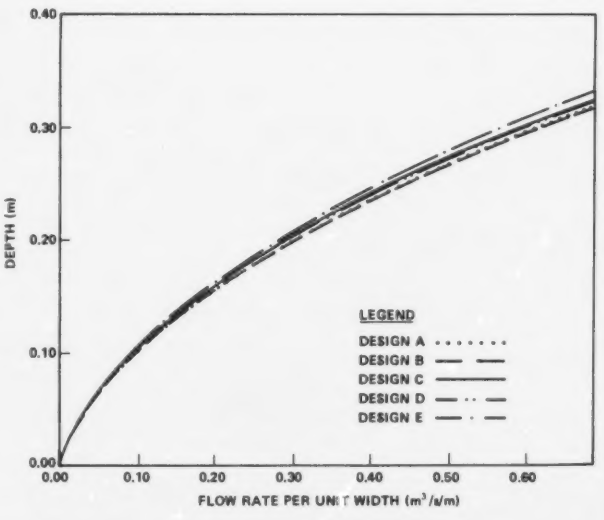
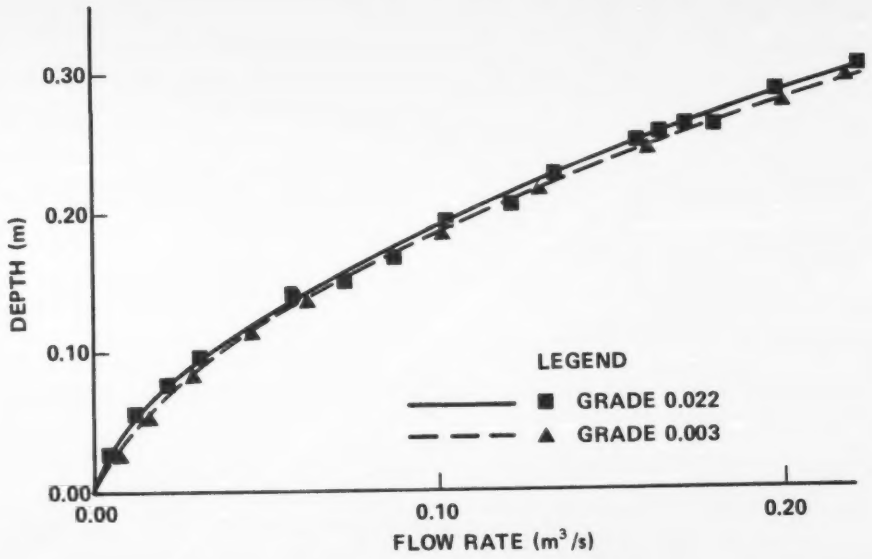


Figure 34/

Capacity Curves for Ditch Inlet Grates Tested, Grate Width = 1.0 m, Grate Slope 2:1



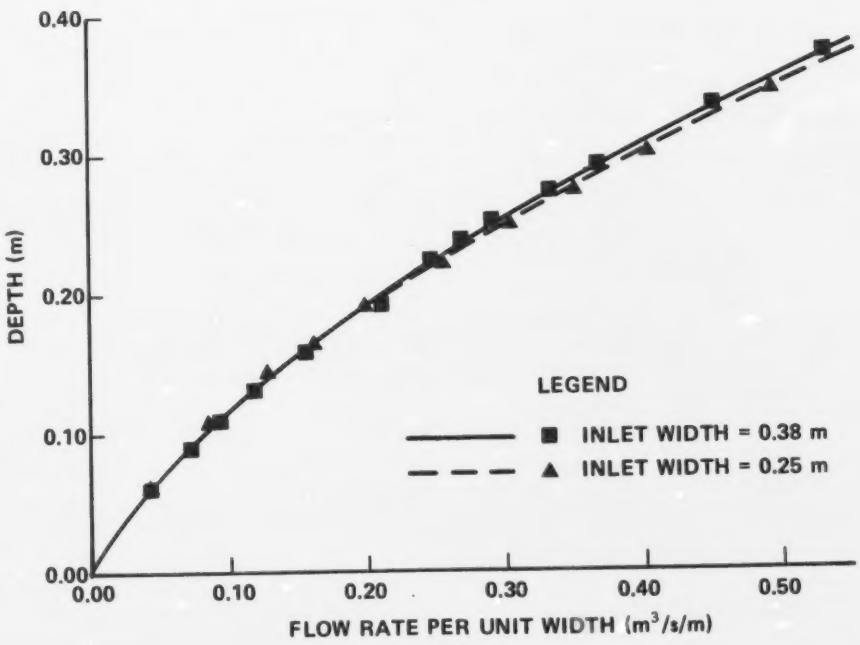


Figure 32/ Effects of Ditch Grade and Inlet Width on Observed Ditch Inlet Capacities

5.4/ Susceptibility of Tested Ditch Inlets to Blockage

The previous section dealt with the hydraulic capacity of ditch inlets without any blockage of inlet openings. Such capacity can be substantially reduced if the inlet openings become partly or fully blocked with debris. Experience shows that ditch inlet blockage occurs in many locations and, in some cases, such occurrences are periodical. The clogging of inlet grates reduces the inlet capacity, and often results in flooding of the adjacent property and in increased maintenance costs for the Ministry. In searching for a solution to the problem of inlet clogging, it was decided to study the susceptibility to clogging of all the inlet designs tested. Towards this end, the clogging of various inlet designs was observed in the laboratory. The results of these observations can be used for qualitative and comparative evaluations of the inlets studied.

Trapping of floating debris by the ditch inlet grates tested is shown in Figure 35.

The test results indicate that although the five grates studied have nearly identical hydraulic characteristics, their susceptibility to clogging varies widely. In general, the results obtained for both grate slopes and two types of debris are quite similar. It can be inferred from Figure 35 that in the tests described here, the grates' susceptibility to clogging was primarily affected by the arrangement of bars grate. The larger the bar spacing, the lower the susceptibility to clogging. The longitudinal bars trapped less debris than the crossbars. The worst performance was observed for the existing grate design (type DD-710, referred to here as the honeycomb grate).

It was also noticed that susceptibility to clogging was not significantly affected by the grate slope, nor by the type of debris used in these tests.

In summary, it can be concluded that the five grate designs tested offer similar hydraulic capacities. The highest capacity was observed for the widely-spaced longitudinal bars. Capacities of the honeycomb grate and of the widely-spaced crossbars were only slightly worse. From the point of view of clogging, the existing design (honeycomb) is the worst, followed closely by closely-spaced crossbars and longitudinal bars. The best results were obtained for the widely-spaced longitudinal bars.

It is therefore recommended to adopt the grate with longitudinal bars spaced 127 mm for future design applications. Where bar spacing of 127mm is not acceptable, a smaller spacing could be adopted—recognizing that by reducing the spacing, the susceptibility of the inlet to clogging increases dramatically. In the tests reported here, reduction of the spacing from 127 mm to 63.5 mm increased the amount of material trapped by the grate by about six times. The reduced-spacing design was only slightly better than the existing honeycomb design.

The grate slope affected inlet capacity. For the milder slope (2:1), lower capacities were observed. It can be expected that further reductions in the grate slope would lead to further reductions in inlet capacity. Within the scope of this study, inlet susceptibility to clogging was not affected by the grate slope.

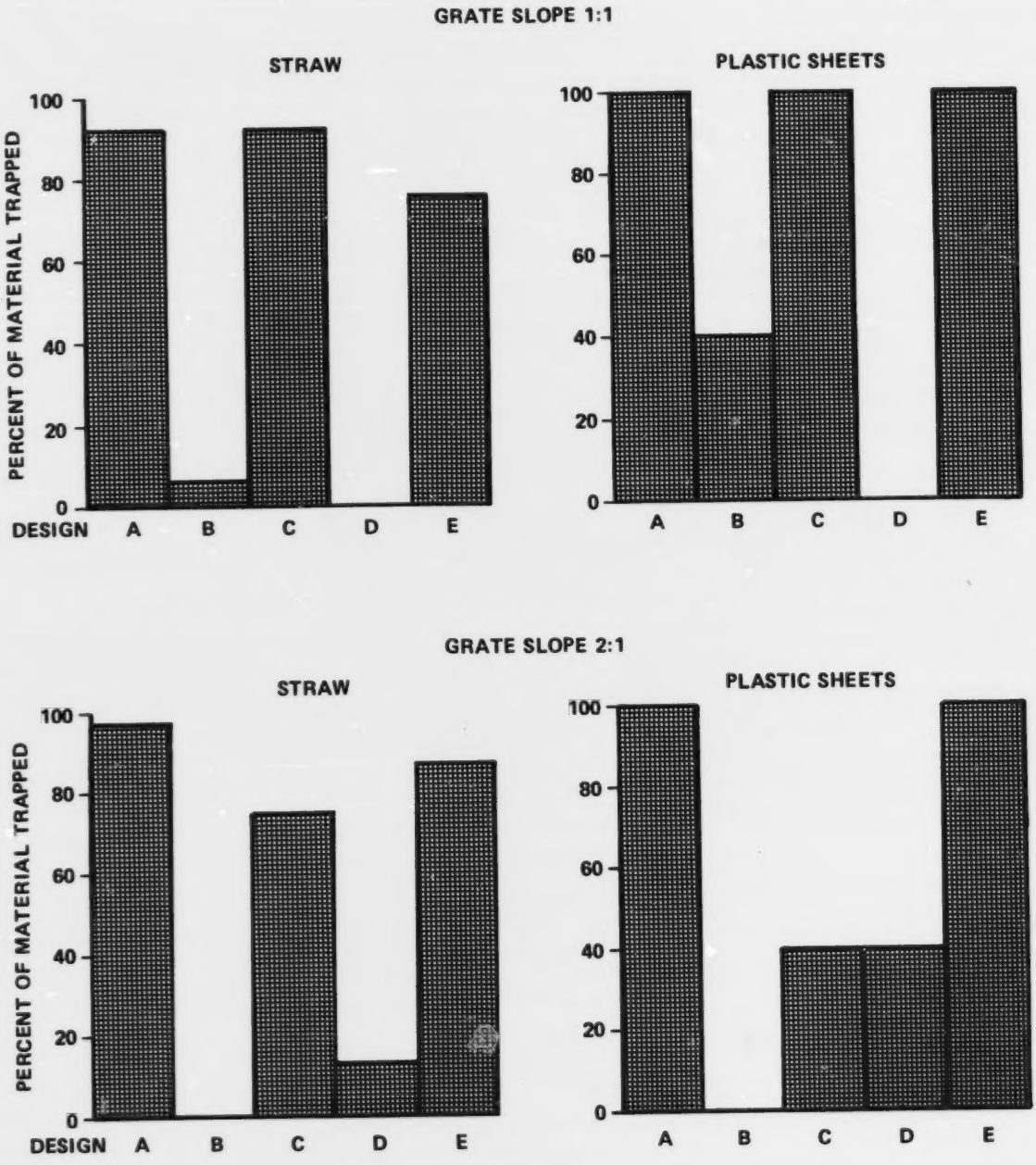


Figure 35/ Trapping of Floating Debris by Ditch Inlet Grates Tested

6/ Bridge Deck Drains

6.1/ Overview of Bridge Deck Drainage

The U.S. Transportation Research Board recently completed a report on "Bridge Drainage Systems," *Synthesis of Highway Practice 67* [17]. Some of the reported findings were used in the study described here and, consequently, a summary of the U.S. report is presented below.

"Bridges should have adequate cross-slope and grade to allow the water to run quickly to the drains. Where grades permit, drains are not used on short bridges but all the water is carried to catch basins at the ends. Bridge drains are sometimes open holes through the deck that can have short pipes to carry the water clear of the beams. More often, however, an inlet box is used to collect the runoff. The spacing and location of drains depend on the amount of rainfall expected, the design of the bridge, the grades, and what is beneath the bridge.

Debris can be controlled by keeping it out of the inlet boxes, accepting and storing it so it cannot go through the system, or transporting it through. Because all debris cannot be kept out, many drainage systems are designed to trap larger debris and let the smaller debris pass. The grates used to screen out larger debris should be hydraulically efficient, strong enough to support traffic, and bicycle-safe. If pipes have adequate size, slope, and curvature for debris control, hydraulic considerations seldom limit the flow.

Maintenance at regular intervals is the key to success of a drainage system. Because this periodic attention is necessary, the design should make it as easy as possible. Numerous cleanouts should be provided in locations where they are easily and safely accessible. Cleaning equipment ranges from shovels to high-pressure water. Recent innovations include a system for backflushing with high-pressure air and a truck-mounted, high-pressure water system.

Disposal of runoff water can be a simple straight drop onto the land or water beneath the bridge or a pipe system to carry the water to a local sewer system. Provisions for controlling or containing spills of hazardous materials are costly and warranted only where the risks are high.

The concensus of current practices indicates that deck cross-slope and grade should be no less than 2% and 0.5%, respectively; that bridge drains may be holes through the deck, fabricated inlet boxes, or catch basins at the ends of the bridge; that inlet areas should be as large as possible; that pipes should have a minimum diameter of 150 mm, a minimum radius of 450 mm, and a minimum slope of 2% (preferably 8%); that cleanout plugs and elbows should be easily accessible; that there should be improved communication between designers and maintenance personnel; and, most importantly, that bridge drainage systems should be regularly and carefully inspected and serviced."

6.2/ Experimental Study of Bridge Deck Drains

Experimental investigations of drains dealt with two aspects of drain operation—the hydraulic capacity and the susceptibility to blockage. The results are presented in the same order.

Bridge deck drains function in the same manner as sewer inlets discussed earlier. The hydraulic capacity of a drain depends on its geometry, installation, and the characteristics of the road surface flow. On flat grades, the drain capacity may be controlled more by the road discharge (i.e., the supply) than by the drain geometry. Drains are typically installed in a small depression to increase their capacities.

Drain hydraulic capacities can be determined by various methods. Among the approaches used in the past, one could name analytical calculations and scale model studies. Analytical calculations require numerous approximations which may lead to large uncertainties in the calculated results. A typical calculation of the drain capacity is based on an assumption that, for drains of a sufficient length, all the water passing over the width of the drain is intercepted. Consequently, the designer determines the road discharge over the drain and then calculates the drain length as a function of the road flow velocity [17]. Such a procedure has two inherent shortcomings—the drain length will vary from case to case and the assumption of a 100% flow interception by drain is violated by all the drains tested.

In scale model studies, it is practically impossible to achieve hydraulic similitude because of small flow depths and spacings between the grate bars. It is recognized, therefore, that actual phototype tests of drains provide the best evaluation of the drain capacity [17].

Hydraulic capacities of bridge deck drains can be substantially reduced if drain openings are blocked by debris. The susceptibility to blockage of the existing drains was, therefore, also studied in the laboratory.

6.2.1/ Experimental Apparatus and Procedures

The road drainage test apparatus built in the Hydraulics Division Laboratory was used for full-scale testing of bridge deck drains. The details of this apparatus were given in Section 4.1.

For individual tests, the roadway grade and crossfall were set. The discharge through the facility was increased in a number of steps until either a pavement spread of 3 m was reached, or the drainage channel was filled to the top of the curb. For each test, the road flow rate, depth and spread (upstream of the drain), and the drain discharge were measured.

In comparative tests of drain blockage, two types of debris were used—straw, and plastic sheets (about 0.2 x 0.2 m). The tests were done for two road flow rates—the low flow of about 0.005 m^3/s and the high flow of 0.025 m^3/s. The road grade was 0.02 m/m and the crossfall was 0.02 m/m.

In the experiments with straw, a known quantity of dry straw was floated on the water surface well upstream of the drain. The straw trapped on the drain grate was collected, dried and weighed to determine the amount trapped.

In the tests with plastic sheets, five plastic sheets (0.2 x 0.2 m) were placed on the water surface upstream of the drain and the number of sheets entrapped by the drain grate was recorded.

Both types of experiments were repeated twice to verify the repeatability of results.

6.2.2/ Experimental Programme

The scope of the experimental programme was established by MTC in consultation with the Hydraulics Division. The total number of experiments was reduced by interpolations and extrapolations of observed data. In all such procedures, an accuracy of $\pm$ 10% had to be maintained.

Two basic types of drains were tested — the drains adjacent to the barrier (in this case a curb) and the median drains. The experimental programme comprised close to 100 runs. A summary of the programme is given in Table 5.

6.2.3/ Hydraulic Capacities of Tested Drains

A preliminary analysis of experimental results indicated that some drains produced similar discharges. Consequently, average capacities of all the drains were compared to find out if capacities of two or more drains could be described by a common expression. The results of this analysis are listed in Table 6.

It is apparent from the data in Table 6 that drains SS 9-1A and SS 9-4A, and drains SS 9-2A and SS 9-2B behaved quite similarly and the corresponding experimental results could be grouped together.

Table 5/ Bridge Deck Drains, Experimental Programme

Drain	Curb & Gutter Type	Grade (m/m)	Crossfall (m/m)	Spread* (m)
SS 9 - 1A SS 9 - 2A SS 9 - 2B SS 9 - 4A SS 9 - 6A	E (Installation adjacent to the curb)	0.005, 0.001, 0.02 0.04, 0.06, 0.07	0.02, 0.04, 0.06	0.5 to 3.0 without overtopping curb
SS 9 - 8A SS 9 - 8B	Median Installation	0.005, 0.01, 0.02 0.04, 0.06	0.02	0.5 to 3.0

*For curb-side installation, the spread was measured from the curb. For median installation, the spread was measured from the waterline to waterline.

Table 6/ Relative Discharges of Tested Drains

	Drain				
	SS 9-1A	SS 9-2A	SS 9-2B	SS 9-4A	SS 9-6A
Discharge in multiples of the mean discharge of the set	0.62	1.45	1.53	0.64	0.76

6.2.3.1/ Drains SS 9-1A and SS 9-4A

Drain SS 9-1A (see Figure 36) is formed by a 219 mm OD pipe whose opening is slightly depressed below the projected road surface. The pipe opening is protected by two square bars (20 x 20 mm) installed parallel to the flow direction. The drain is a bicycle-safe design.

In an overall evaluation drain SS 9-1A is rather simple and inexpensive. It has a relatively small opening for intercepting the road flow (the free-opening area is 0.0252 m^2) and, consequently, it has a fairly low capacity. As follows from the comparison of the drains tested, drain SS 9-1A has the lowest capacity among all the drains, followed closely by drain SS 9-4A. In fact, the difference in mean capacities of these two drains was less than 4%.

Drain SS 9-4A (Figure 36) is formed by a 219 mm OD pipe (the same as SS 9-1A) which is inclined at 45° and directs water away from the bridge structure. Such an arrangement appears to have two advantages—the opening area (which is an ellipse obtained by cutting the 219 mm OD pipe at 45°) is larger, and drainage water is likely to discharge further from the bridge structure.

The free-opening area of drain SS9-4A is 0.0314 m^2. Although this area is significantly larger than that of drain SS 9-1A, the former drain is rather shallow on the side closer to the road centre and, consequently, does not intercept significantly higher flows than drain SS 9-1A. As stated before, drain SS 9-4A has a capacity of only 4% larger than that of drain SS 9-1A.

In an overall evaluation, drain SS 9-4A is relatively inexpensive and has a fairly low capacity. Compared to drain SS 9-1A, drain SS9-4A is slightly more elaborate, but may better direct runoff from the bridge structure. Unless this feature is found particularly important by MTC, one could discontinue design SS 9-4A and replace it by a similar drain such as SS 9-1A.

Observed capacities of drains SS 9-1A and SS 9-4A were plotted together in Figure 37. It was noted that drain capacities varied insignificantly with the deck grade and crossfall. Detailed analyses of such variations yielded standard deviations of about $\pm$ 12%, which was quite close to the expected accuracy of the observed data. Under these circumstances, it was decided to simplify the data analysis by considering all the observed capacities, for both drains and various grades and crossfalls, as a single data set. This set was then approximated by a regression equation in the following form:

$$Q_d = 0.000\,13 + 0.099\,14\,d - 0.118\,49\,d^2 \quad (18)$$

where Q_d is the drain capacity (discharge) in m^3/s, d is the depth of flow upstream of the drain, in metres, and the numerical constants were obtained from the regression analysis. Equation 18 is applicable for the condition $0.00 < d < 0.15$ m. For $d = 0$, Q_d is set equal to zero.

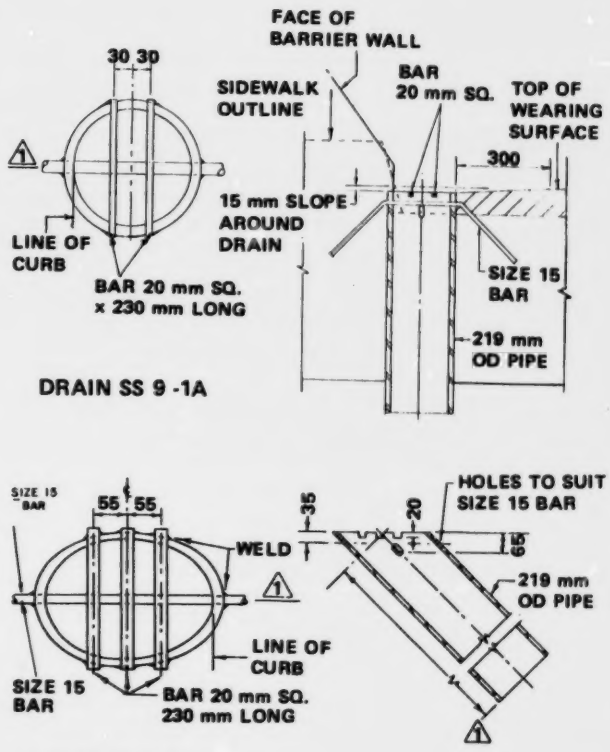


Figure 36/ Drains SS 9-1A and SS 9-4A

It appears from Figure 37 that, for low flow depths, the observed drain discharges tend to fall below the regression curve. This is caused by the fact that, in the region of low depths and flows, the drain discharge is controlled to a large extent by the road flow which supplies water to the drain. On low grades ($S \leqslant 0.02$ m/m), for which most of the tests were done, the road flows are particularly small and this is then reflected in low observed drain discharges. It is possible to account for the control of the drain discharge by the road flow by simply imposing a constraint $Q_{drain} \leqslant Q_{road}$. Such a constraint was used in the design procedure described in Chapter 9. The sole purpose of this constraint is to avoid a physically impossible situation where the drain discharge calculated from the regression equation would exceed the supply (i.e., the discharge conveyed by the road surface).

The tendencies described here for drains SS 9-1A and SS 9-4A were also observed for other drains and taken into consideration as described above.

The main advantage of having a single drain capacity curve, described by a regression equation, will become apparent in Chapter 9 on the design of bridge deck drainage.

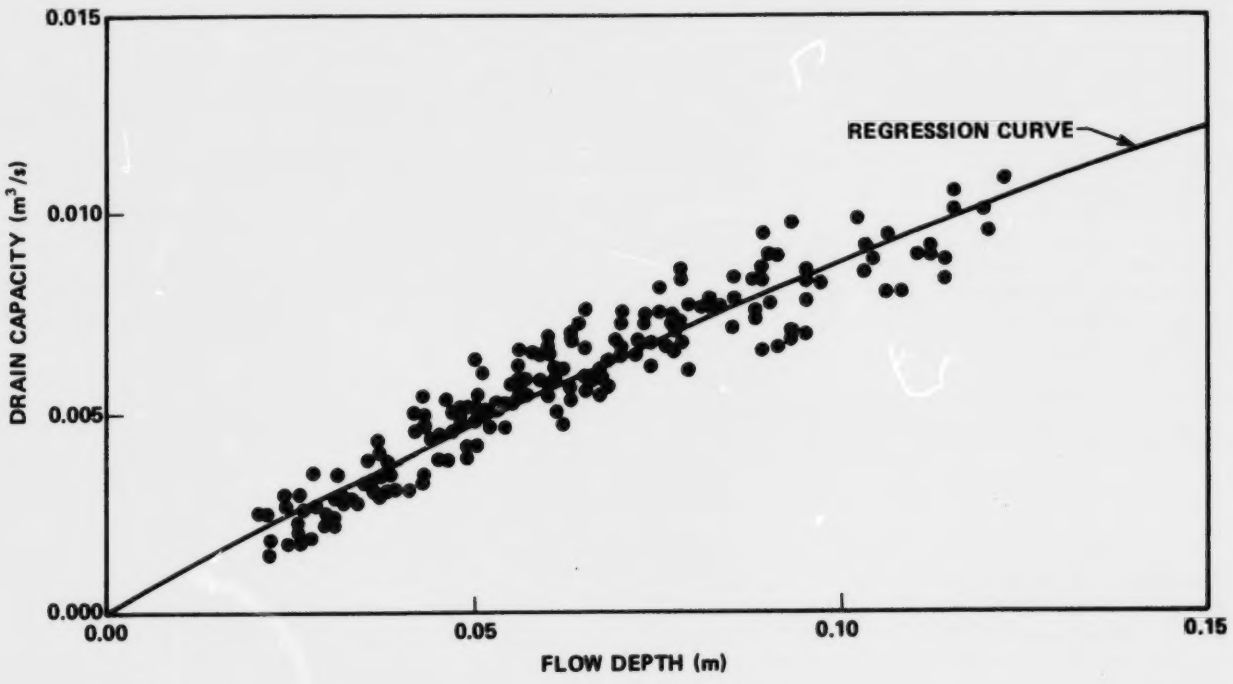


Figure 37/ Hydraulic Capacity Curve for Drains SS 9 -1A and SS 9 -4A

6.2.3.2/ Drains SS 9-2A and SS 9-2B

Drains SS 9-2A and SS 9-2B are shown in Figure 38. Both drains have a rectangular opening 1000 x 230 mm connected to a drain pipe of 219 mm OD. The two drains differ only in the arrangement of the grate bars—type 2A has two longitudinal bars and type 2B has ten crossbars. Only the latter type appears to be bicycle-safe.

Drains SS 9-2A and SS 9-2B have the highest capacities among all the curb drains tested. This follows from their large area of free openings—0.210 m^2 for the type 2A and 0.2024 m^2 for the type 2B. On average, these two drains intercepted more than twice the flow intercepted by other curb drains tested.

The capacities of both drains differed very little—by less than 5%. Surprisingly, the type 2B with crossbars yielded the higher capacity. An explanation for this deviation from the earlier results obtained for sewer inlets [8] was found in the inefficient operation of the drain with longitudinal bars. The flow enters this drain at fairly high velocity and partly deflects upwards at the downstream end and leaves the box again. Note that at the downstream end, the drain box is less than 60 mm deep. In the case of the drain with crossbars, flow velocity gradually decreases as the flow passes over the crossbars and, consequently, a larger part of the flow is intercepted.

Since the average capacities of both drains differed insignificantly, the observed data for both drains were grouped together and plotted in Figure 39. Detailed analyses of capacity variations for various grades and crossfalls indicated that these variations were fairly small, rarely exceeding $\pm$ 15%. To simplify the design procedure, all the data for various grades and crossfalls were grouped together and approximated by the following regression equation:

$$Q_d = 0.000\,39 + 0.268\,d - 0.440\,d^2 \qquad (19)$$

where Q_d is the drain capacity (discharge) in m^3/s, d is the depth of flow upstream of the drain, in metres, and the numerical constants provided the best fit. Equation 19 is applicable for $0.00 < (d < 0.15$ m. For $d = 0$, Q_d is set equal to zero. Drain capacities calculated from Equation 19 are listed in the Appendix. The selection of the above regression equation followed from a condition that the regression curve should pass through or close to the origin. As discussed earlier, the observed points fall below the curve in the region of low depths where the drain capacity is controlled by the road flow discharge. This can be accounted for by imposing the constraint $Q_d \leq Q_{road}$.

In an overall evaluation, drains SS 9-2A and SS 9-2B have the highest capacities among the tested curb drains, but will be more expensive to manufacture and install. When comparing types 2A and 2B, the latter type has a slight advantage, because it is bicycle-safe and has a slightly higher capacity. Where large drain spacings are desirable, drains SS 9-2A and SS 9-2B are the clear choice among the drains tested.

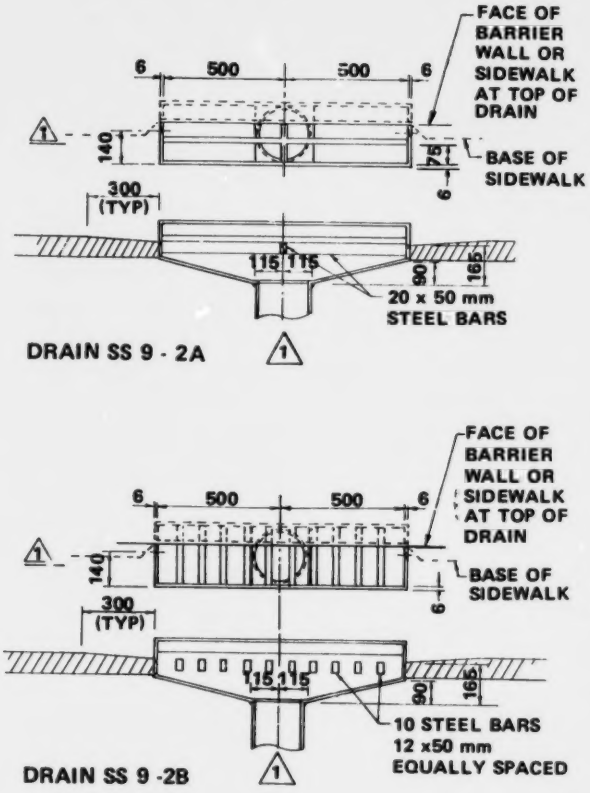


Figure 38/ Drains SS 9 - 2A and SS 9 - 2B

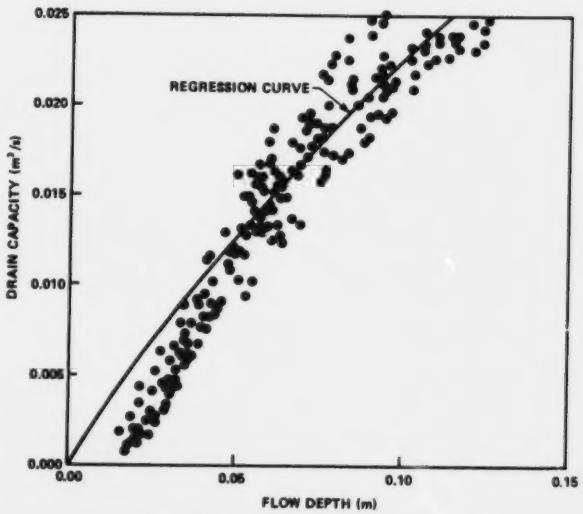


Figure 39/ Hydraulic Capacity Curve for Drains SS 9 -2A and SS 9 -2B

6.2.3.3/ Drain SS 9-6A

Drain SS 9-6A is a special design recommended for use with CSA G40.21-M-350A, *Steel Girders*. The drain is formed by a 200 mm square tube fitted with two longitudinal bars in the opening (see Figure 40). It appears to be a relatively simple and inexpensive drain design with a limited capacity. The area of free openings is 0.0320 m^2, slightly larger than that mentioned earlier for circular drains. The drain appears to be bicycle-safe.

Observed drain discharges were plotted in Figure 41. It appears that for practical purposes, all the observations for various grades and cross falls could be approximated by a regression equation:

$$Q_d = 0.000\,18 + 0.131\,d - 0.189\,d^2 \qquad (20)$$

where Q_d is the drain capacity (discharge) in m^3/s, d is the depth of flow upstream of the drain, in metres, and the numerical constants were obtained from the regression analysis. Equation 20 is applicable for $0 < d < 0.15$ m. For $d = 0$, Q_d is set equal to zero. Drain capacities calculated from Equation 20 are listed in the Appendix.

In an overall evaluation, drain SS 9-6A, which is a special design used with certain bridge structures, is a simple, inexpensive design of limited capacity.

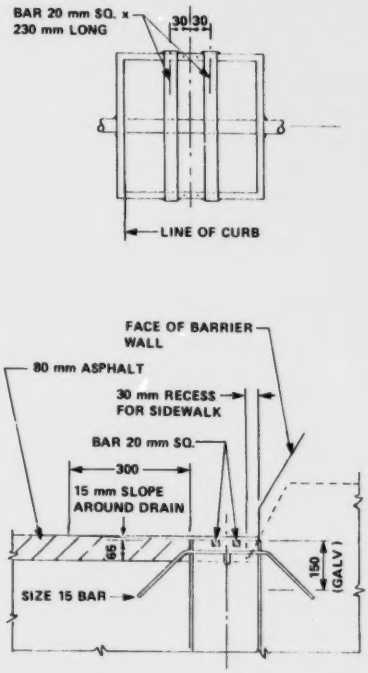


Figure 40/ Drain SS 9-6A

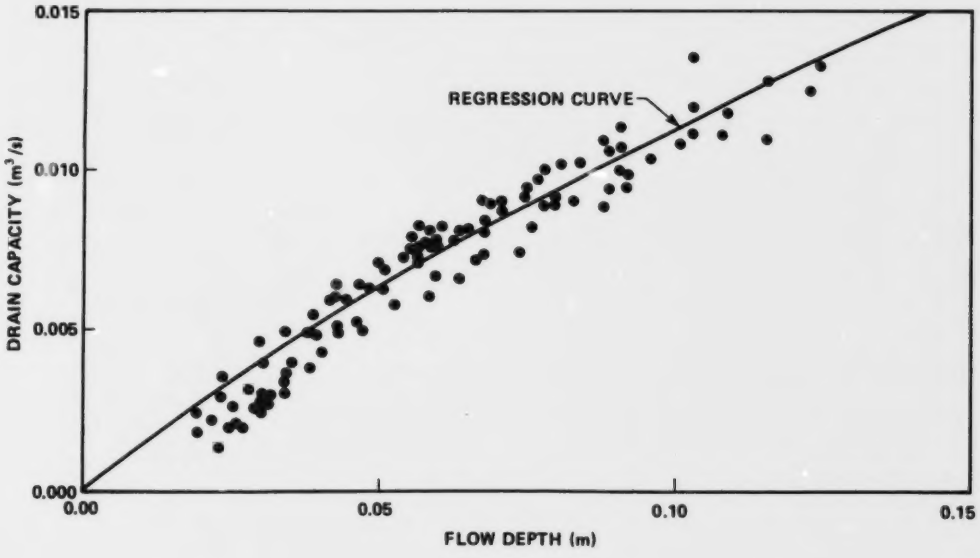


Figure 41/ Hydraulic Capacity Curve for Drain SS 9 -6A

6.2.3.4/ Median Drains SS 9-8A and SS 9-8B

Drains SS 9-8A and SS 9-8B were the only median drains studied. Basically, these two drains are identical to the earlier-discussed types SS 9-2A and SS 9-2B, except for the installation (see Figure 42). The former drains are installed in the median, the latter adjacent to the curb. A median installation allows a much better inflow of runoff to the drain than the curb installation and this is fully reflected in the observed drain capacities.

The only difference between types 8A and 8B is in the arrangement of the grate bars. The 8A type has longitudinal bars, whereas the 8B type has crossbars.

Observed capacities of drains SS 9-8A and SS 9-8B were analysed in the same way as described for other drains—the data obtained for both types and various grades and crossfalls were grouped together and approximated by the following regression equation:

$$Q_d = 0.0009 + 0.53\,d - 1.47\,d^2 \qquad (21)$$

where Q_d is the drain capacity in m^3/s and d is the depth of flow upstream of the drain, in metres. Equation 21 is applicable for $0 < d < 0.15$ m. For $d = 0$, Q_d is set equal to zero. Drain capacities calculated from Equation 21 are listed in the Appendix. For depths smaller than 0.015 m, the calculated capacities were slightly reduced to force the rating curve through the origin.

In an overall evaluation, median drains SS 9-8A and SS 9-8B have the highest capacities among all the tested drains.

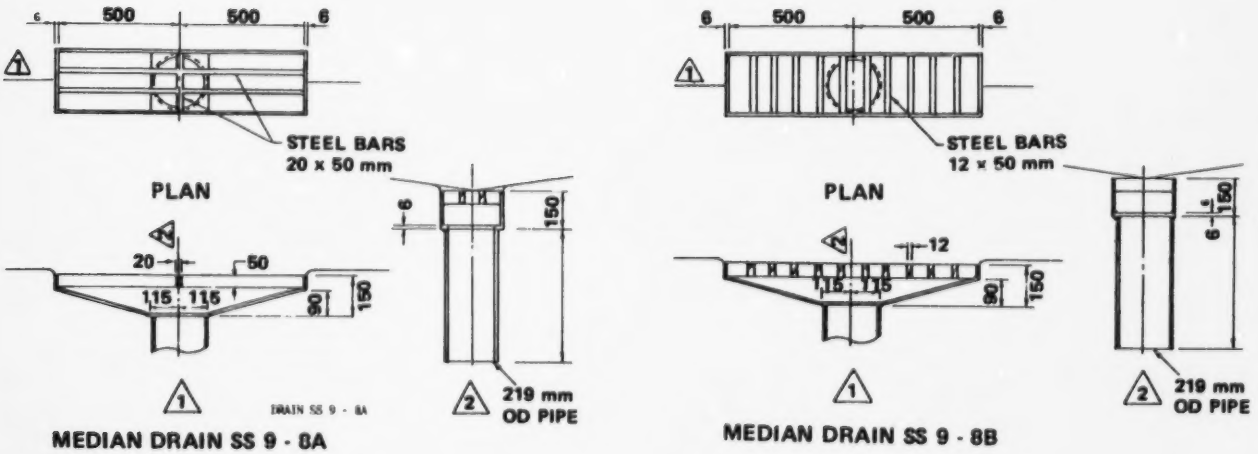


Figure 42/ Median Drains SS 9 - 8A and SS 9 - 8B

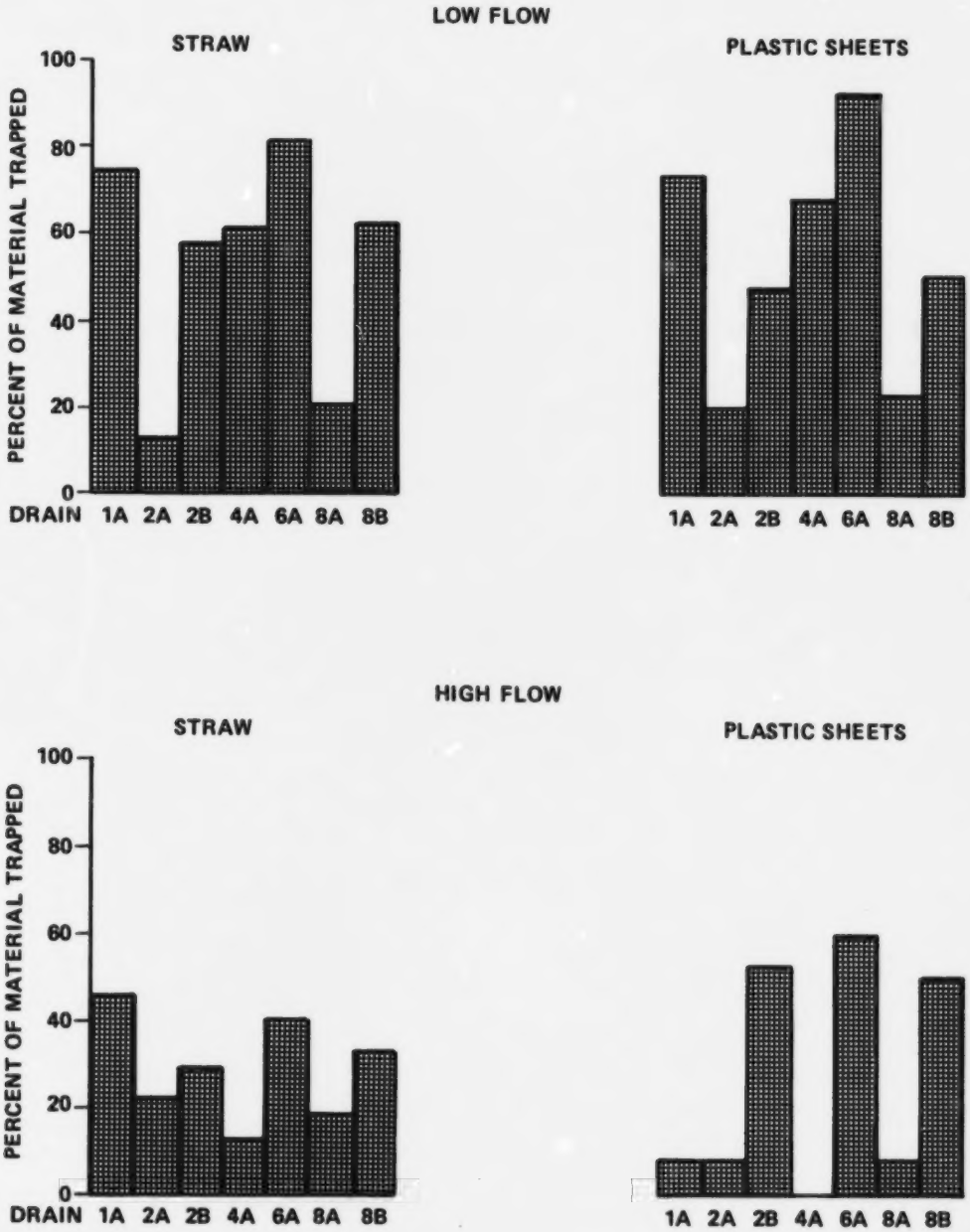


Figure 43/ Trapping of Floating Debris by Drains Tested

Table 7/ Drain Susceptibility to Blockage

	Drain						
	SS 9-1A	SS 9-2A	SS 9-2B	SS 9-4A	SS 9-6A	SS 9-8A	SS 9-8
Mean Percentage of Debris Trapped	50	16	47	35	69	17	49
Susceptibility to Blockage (Ascending Order)	6	1	4	3	7	2	5

Table 8/ Characteristics of Bridge Deck Drains

	Drain						
	SS 9-1A	SS 9-2A	SS 9-2B	SS 9-4A	SS 9-6A	SS 9-8A	SS 9-8
Relative Costs	Low	High	High	Low	Low	High	High
Relative Capacity	Low	Medium-High	Medium-High	Low	Low	High	High
Bicycle-Safe	Yes	No	Yes	Yes	Yes	No	Yes
Susceptibility to Blockage	Medium	Low	Medium	Low-Medium	High	Low	Mediu
Special Features	—	—	—	Directs discharge away from the structure	To be used with CSA steel girders	Median drain	Media drain

6.2.4/ Susceptibility to Blockage of Tested Drains

The hydraulic capacity of bridge deck drains can be substantially reduced if drain openings become partly or fully blocked with debris. A reduced drain capacity then may lead to ponding and the resulting adverse effects on traffic and increased maintenance costs. It is, therefore, of interest to evaluate the susceptibility to blockage of the drains tested and to look for possible improvements. Such an evaluation was based on laboratory observations of the trapping of floating debris by the tested drains. Average values of debris quantities trapped by the tested drains are shown in Figure 43.

Several tendencies can be inferred from the data in Figure 43. It is quite apparent that, among the drains tested, drains SS 9-2A and SS 9-8A are the least susceptible to blockage. On average, these drains trapped less than 16% of the incoming debris. The remaining debris either passed through the drain, for low flows, or bypassed the drain, for high flows. It was also noted that drains SS 9-2A and SS 9-8B performed much better than similar designs with crossbars (SS 9-2B and SS 9-8B) which trapped about three times more debris.

With the exception of drains SS 9-2A and SS 9-8A, the susceptibility to blockage of the tested drains depended on the road flow rate. At low flows and spreads, a large fraction of the road flow was intercepted and this resulted in higher amounts of trapped debris. At high flows, most of the flow bypassed the drains and so did the debris.

Finally, the mean percentages of debris trapped (representing the susceptibility to blockage) are listed in Table 7.

6.2.5/ Evaluation of Tested Drains

Basic characteristics (i.e., the hydraulic capacity and susceptibility to blockage) of the drains tested are given in Table 8. It follows from this table that, where high drain capacities are required in curb installations, drains SS 9-2A and SS 9-2B should be used. When comparing these two drains, only the type 2B is bicycle-safe, but it is more susceptible to blockage by debris. The remaining three drains, used in curb installations, should be used in locations where closely spaced, inexpensive drains are acceptable.

Both median drains have fairly high capacities; however, the type SS 9-8A with longitudinal bars is less susceptible to blockage than the crossbar design (SS 9-8B).

6.3/ New Bridge Deck Drain Designs

One of the study objectives was to evaluate the currently-used drain designs and, if feasible, to propose new designs of better characteristics.

Regarding drain capacities, the drains tested (with the exception of drains SS 9-2A and SS 9-2B) have relatively low capacities. Improvements in drain capacities would be desirable and could be achieved by increasing the drain width from the existing values of 205 mm to 230 mm, to 300 mm. It is understood that such a modification is acceptable to MTC. In the absence of experimental data for new designs, one can only estimate the capacities of newly-proposed designs.

A tentative proposal for new drains derived from drains SS 9-1A, SS 9-4A, and SS 9-6A is shown in Figure 44. The width of these designs is 300 mm. To make the designs bicycle-safe, additional bars had to be added to the original grate. It is estimated that these new drains would have capacities about 50% higher than the old designs. Note that these increased capacities would still be below those of the existing drains SS 9-2A and SS 9-2B.

Drains SS 9-2A and SS 9-2B could be also modified by increasing their width from 230 mm to 300 mm and by increasing the depth of the drain box. A tentative proposal of the modified design is shown in Figure 45. It is expected that the modified designs would have capacities about 30% higher than the existing ones.

Regarding susceptibility to blockage, there appears to be little room for further improvements of the existing drain grates. Comparative tests of drains SS 9-2A and SS 9-2B indicated that longitudinal bars were less susceptible to blockage than the crossbars. With the exception of drain SS 9-2B, all the drains have longitudinal bars, fairly widely spaced, and further improvements do not appear to be feasible.

Where bicycle safety is important, drains with crossbars (or diagonal bars) may have to be used, recognizing that this will lead to an increased probability of drain blockage.

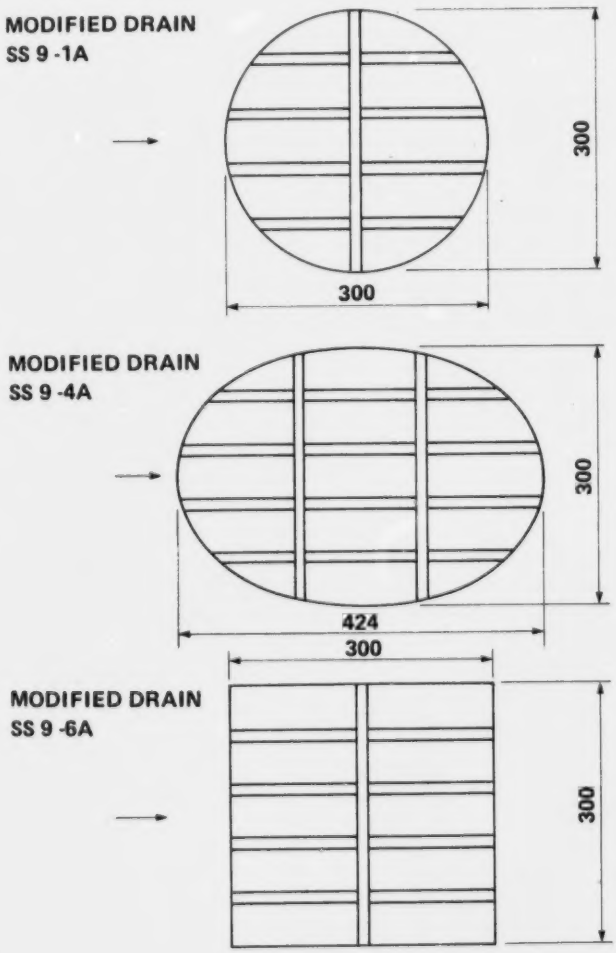


Figure 44/ New Drain Designs Derived from Drains SS 9 -1A, SS 9 -4A, and SS 9 -6A

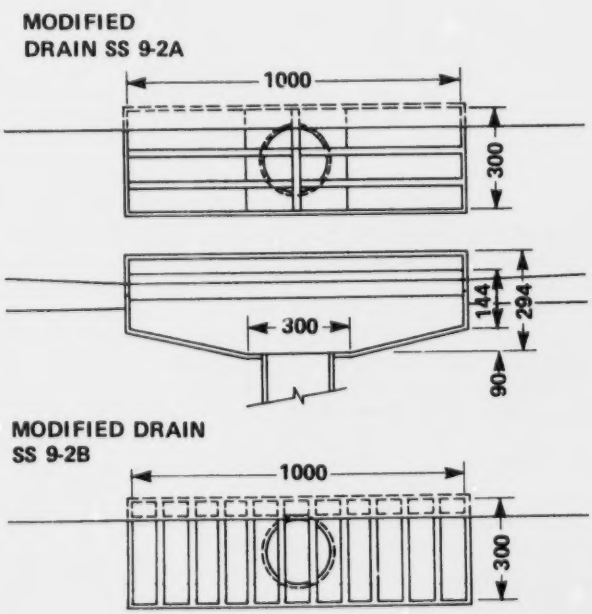


Figure 45/ New Drain Designs Derived from Drains SS 9-2A and SS 9-2B

7/ Summary of Laboratory Study Findings

The principal findings of the laboratory study of inlets and drains are summarized below.

Throughout this study, inlets and drains were considered as a part of the gutter-inlet system, and all the tests were conducted for a quasi-uniform flow upstream of the inlet.

7.1/ Inlets on a Grade

The hydraulic capacity of grate inlets on a grade is affected by the grate geometry and by characteristics of the gutter flow.

The characteristics of the surface drainage facilities which affect the gutter flow and, consequently, the inlet capacity were studied in detail and included the following: curb and gutter geometry, road grade, road crossfall, and spread. All the tests were done for one surface roughness characterized by the Manning n = 0.013. For uniform flow, the aforementioned parameters define the gutter flow rate.

Observed inlet capacity curves could be divided into two distinct parts. In the first part, corresponding to low grades ($S < 0.02$ m/m), the inlet capacity was controlled by the gutter flow rate, which limited the water supply to the inlet. With increasing grade, the gutter flow rate increased and, consequently, the inlet capacity increased as well. In the second part, which corresponded to steep grades ($S > 0.02$ m/m), the gutter flow velocity started to interfere significantly with the inflow into the inlet. Although the gutter flow rates and the depths of flow over the grate further increased, the inlet discharge remained about constant, or even slightly decreased in the case of very steep grades.

Inlet capacities generally increased with increasing flow spreads and road crossfalls, since increasing spreads and crossfalls resulted in increased depth of flow over the inlet grate.

The geometry of curb and gutters affects the gutter flow and, consequently, the inlet capacity as well. The curb height controls the maximum attainable spread (i.e., without curb overtopping) and also the maximum depth of flow over the grate. It was noted that curb type B was the highest (h = 150 mm), followed by curb type A (h = 125 mm), and type curb D (h = 100 mm). The maximum spreads and inlet capacities corresponding to the curb-full flow (i.e., the water surface flush with the top of the curb) were ranked in the same order.

Gutters play an important role in road drainage. They increase the depth of flow over the inlet grate and, consequently, the inlet capacity as well. For identical spreads, inlets installed on roads with gutters had capacities larger than inlets installed without gutters.

Grates DD-713-A (with bars parallel to the curb) and DD-713-B (the bicycle grate) may be installed with either curb and gutter type A or type B. Installations of both inlet grates with the curb and gutter type A yielded slightly larger (by 4%) capacities than those with curb and gutter type B. This followed from the fact that, for identical spreads, curb and gutter type A has a slightly larger flow area than curb and gutter type B.

Comparisons of the capacities of grates DD-713-A and DD-713-B showed that grate DD-713-A had, on the average, capacities larger by 8% than those observed for grate DD-713-B. The superior performance of grate DD-713-A was particularly marked for steep grades, and large crossfalls and spreads. Under those conditions, grate DD-713-A induced less disturbance to the flow and yielded capacities 20% to 25% larger than those observed for grate DD-713-B.

The total area of openings of grate DD-713-A exceeded that measured for grate DD-713-B. For both grates, the central supporting crossbar was a source of flow disturbances and reduced the grate capacities.

Grate DD-705 behaved similarly to grates DD-713-A and DD-713-B. Its capacity rapidly increased with increasing grade in the region of relatively low grades ($S < 0.02$ m/m). For steeper grades ($S > 0.02$ m/m), the capacity hardly changed with varying grade.

Twin inlets on a grade were tested for grates DD-713-A and DD-705. In both cases, the capacity of the twin inlet exceeded, on average, the capacity of the corresponding single inlet by about 20%. This increase varied from a few percent at low spreads and grades to about 40% at large spreads. It is conceivable that the twin-inlet DD-713-B would behave similarly. The economic effectiveness of twin inlets on a grade is questionable because, under typical design conditions, the cost saving due to the larger capacity of twin inlets hardly compensates for the larger construction costs of twin inlets. Consequently, the use of twin inlets should be restricted to situations where the cost-effectiveness of twin inlets can be achieved. Such a case may be the use of twin inlets in a sag, where the capacity increase over a single inlet varies from 35% for low depths, to about 100% for large depths.

In addition to the above MTC grates, two municipal grates, the Kitchener-Waterloo-Cambridge (KWC) grate and a round frame (RF) grate, were also tested. The KWC grate is a bicycle-proof grate with an above average area of openings. It can be used advantageously in a sag where it yields the highest capacity of all the grates tested (for depths > 0.20 m). On low grades, the KWC grate has similar capacities to other MTC grates (DD-713-A and -B). For steeper grades, the KWC grate has smaller capacities than the MTC grates. This capacity reduction is caused by three crossbars which interfere with the flow over the grate. The extent of this interference increases with increasing grade and gutter flow velocity.

The round frame inlet was developed to reduce manufacturing and maintenance costs. From the hydraulics point of view, the RF grate offers, on low grades and in a sag, capacities comparable to those reported for the MTC grates. On steeper grades ($S > 0.01$ m/m), the RF grate capacity starts to fall significantly (10% to 30%) below that of other MTC grates.

Several new inlet grates were tested for the grades of 0.003 m/m and 0.02 m/m. In this range of grades, a large percentage (70% to 95%) of the flow over the grate was intercepted and, consequently, the capacities of new designs proved to be not very sensitive to the characteristics of the grate. Comparative testing of new designs revealed several weak tendencies summarized in the following. On the low grade, longitudinal bar grates performed slightly better than diagonal bar grates. The capacity of the longitudinal bar grate was improved by removing or depressing the crossbar and by increasing the bar spacing from 50 mm to 77 mm. The rounding-off of bar edges had no noticeable effect on inlet capacity. It appears that, on low grades, the existing grates operate fairly efficiently and hardly any improvement can be achieved by the grate modifications studied.

One combination inlet, type C-93, was also tested on a grade. The C-93 inlet had capacities about 30% lower than similar MTC grate inlets. These lower capacities followed from the fact that the C-93 inlet has a small area of grate openings (0.113 m^2 compared to 0.145 m^2 for others), and the curb opening contributes very little to the total intercepted flow. The combination inlet may offer some advantages in a sag installation where the probability of grate blockage is increased. If the grate is clogged, the curb opening offers some relief capacity to reduce potential flood damages.

7.2/ Inlets in a Sag

The capacity curves of inlets in a sag could be approximated by a rectangular weir equation for small depths and by an orifice equation for larger depths (say, $d > 0.20$ m). A good agreement between the theoretical and experimental curves was obtained only for the weighted-mean depths over the grate and for a coefficient of discharge $C = 0.62$ adopted in the orifice equation. It follows from these findings that the inlet capacity in a sag is controlled, for small depths, by the effective inlet perimeter and, for larger depths, by the area of grate openings. Consequently, inlets with identical perimeters and areas of openings have identical capacity curves. This was confirmed by experimental data collected for inlets DD-713-A, DD-713-B, and DD-705.

The KWC grate has a perimeter identical to that of the three aforementioned grates, but a larger area of openings. Consequently, the KWC capacity curve follows that found for the aforementioned three inlets in the region of small depths. For large depths, the KWC grate had capacities larger in proportion to its larger area of openings.

Finally, a twin DD-713-A inlet was also tested in a sag installation. The effective perimeter of a twin inlet is only 38% larger than that of the single inlet. The capacities of the twin inlet, observed for small depths only, exceeded the capacities of the single inlet by about the same percentage. Conceivably, for larger depths, the twin inlet capacity would be about twice the single inlet capacity.

7.3/ Ditch Inlets

Five new and existing designs of ditch inlet grates were tested. The grates tested could be described as *honeycomb, longitudinal bar,* and *crossbar* designs with various spacing. From the hydraulics point of view, all the designs operated quite efficiently and typically conveyed from 85% to 90% of the free-opening discharge. The second objective of this study task was to evaluate the susceptibility of all the designs to clogging. For this criterion, large differences in performance were observed for individual inlets. The existing design (a honeycomb DD-710) was most susceptible to clogging followed by closely-spaced (63.5 mm) crossbars and longitudinal bars. The grate with widely-spaced (127 mm) longitudinal bars was least susceptible to clogging, followed by the widely-spaced crossbars.

7.4/ Bridge Deck Drains

Hydraulic capacities of seven bridge deck drains currently used by MTC were established by full-scale testing in the hydraulics laboratory. Drain capacities depended on the drain geometry and on the road flow conditions.

Among the five drains adjacent to the curb, only the types SS 9-2A and SS 9-2B had satisfactory capacities—typically ranging from 0.006 to 0.013 m^3/s for depths of flow from 0.02 to 0.05 m. The corresponding capacities of the remaining three drains varied from 0.002 to 0.006 m^3/s.

The observed drain capacities varied primarily with varying depth of flow; the effects of the road grade and crossfall were practically negligible.

Among the drains tested, the type SS 9-2A, which has widely-spaced longitudinal bars, was by far the least susceptible to blockage by debris. Unfortunately, this drain is not bicycle-safe.

Two median drains, SS 9-8A and SS 9-8B, were also tested. These drains were quite similar to the types SS 9-2A and SS 9-2B discussed earlier and produced similar results.

8/ Design Procedure for Road Drainage Inlets

8.1/ General Description

One of the main objectives of the study was to develop a new procedure for road drainage design using the results of laboratory tests of inlets. Various techniques of implementing the research results in the design procedure were discussed with MTC personnel. The design procedure adopted combines the use of simple formulae and design charts. It is believed that such a procedure will best serve the needs of the Ministry.

A notation sketch for the design procedure described below is shown in Figure 46. For the drainage design, the following information is given.

		Dimension
The roadway width	W_p	m
The paved shoulder width	W_s	m
The curb and gutter type		
The total drained width (W_g = the width of gutter)	$W = W_p + W_s + W_g$	m
The grade (longitudinal slope)	S_o	m/m
The crossfall	S_x	m/m
The type(s) of inlets to be used		
The type of pavement and the corresponding Manning's roughness	n	
The spread	T	m
The runoff coefficient	C	
The concentration time	t_c	min
The return period of the design rainfall	R	years
The average design rainfall intensity for the concentration time t_c and the return period R (this value is read from a rainfall atlas).	i_{t_c}	mm/h

From the above data, one can calculate:

- the runoff peak flow (as a function of the drained area), and — Q_p — m^3/s
- the maximum gutter flow capacity corresponding to spread T. — Q_{gT} — m^3/s

Finally, the design is completed by calculating the following two parameters:

- the location of the first inlet (measured from the upper end of drainage area), and — L_1 — m
- the spacing for the consecutive inlets. — L_c — m

A description of the individual steps of the proposed design procedure follows.

8.2/ Gutter Flow — Design Considerations

Using the procedure described in Chapter 3, a series of design charts was prepared for flow in various types of gutters. In all calculations, it was assumed that the drainage channel roughness could be described by the Manning's coefficient n = 0.013.

In general, the gutter flow can be described by the following equation:

$$Q_g = K\underline{f}(G, S_x, T, S_o) \qquad (22)$$

where K is a constant, and G is a factor describing the curb and gutter geometry. Among the independent variables in Equation 22, only the grade S_o varies continuously; all the other variables vary in preselected steps. Consequently, in the design charts the gutter flow rate was plotted as a function of the grade, and G, S_x, and T were used as parameters. For each curb and gutter type, a separate design chart was prepared. Such charts contain four families of gutter capacity curves; one family for each crossfall. Each family then comprises a number of curves $Q_g = K'\underline{f}(S_o)$, which were prepared for individual spreads. As specified by MTC, the following spreads were considered:

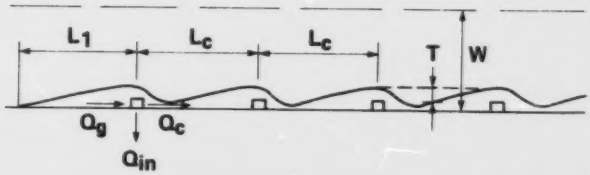
0.50, 0.75, 1.00, 1.50, 2.00, 2.50, 2.70, and 3.00 m.

The gutter capacity curves were plotted only for those spreads which did not cause overtopping of the curb. Consequently, the number of curves in the individual curve families varied from 2 to 8.

The design charts for gutter flow are given in Figures 47 to 50 in the following sequence:

Curb and Gutter Type	Figure No.
A	47
B	48
D	49
E (no gutter)	50

Numerical values of the calculated gutter capacities are given in the Appendix. If the design roughness coefficient n_d differs from n = 0.013, the listed flow capacities have to be multiplied by a correction factor equal to $0.013/n_d$.



LEGEND

L_1 = LOCATION OF THE FIRST INLET
L_c = SPACING OF CONSECUTIVE INLETS
W = DRAINED AREA WIDTH
T = DESIGN SPREAD
Q_g = GUTTER FLOW RATE
Q_{in} = INLET FLOW RATE
Q_c = CARRYOVER FLOW RATE
$Q_c = Q_{in} + Q_c$

Figure 46/ Notation Sketch — Road Drainage

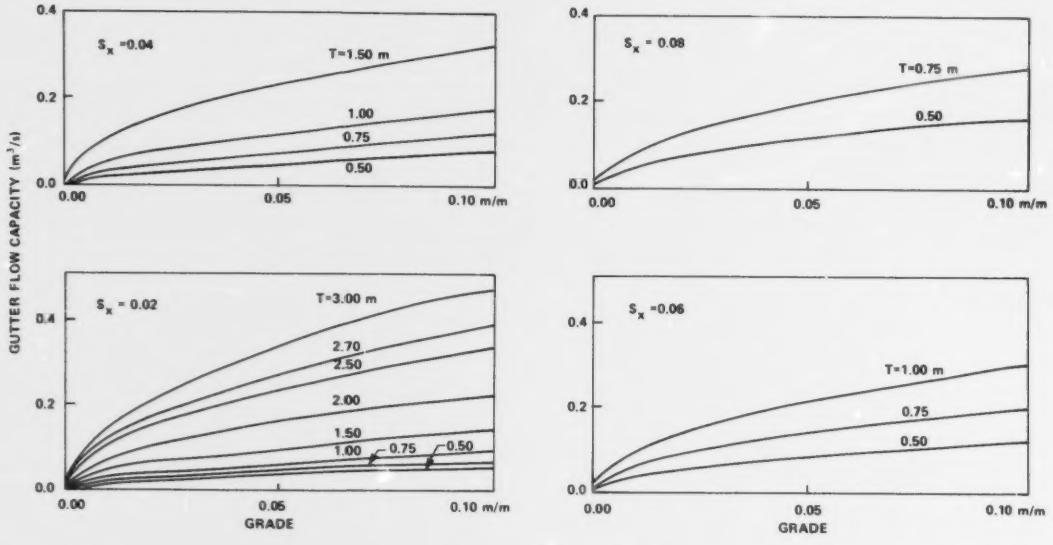


Figure 47/ Gutter Flow Rating Curve: Curb & Gutter Type A

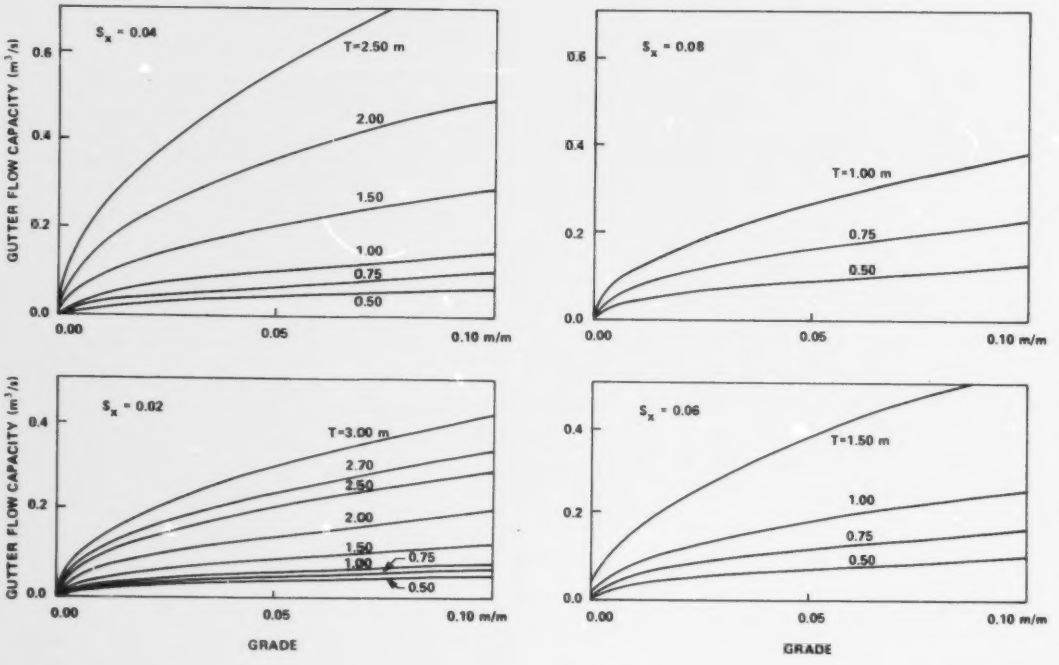


Figure 48/ Gutter Flow Rating Curve: Curb & Gutter Type B

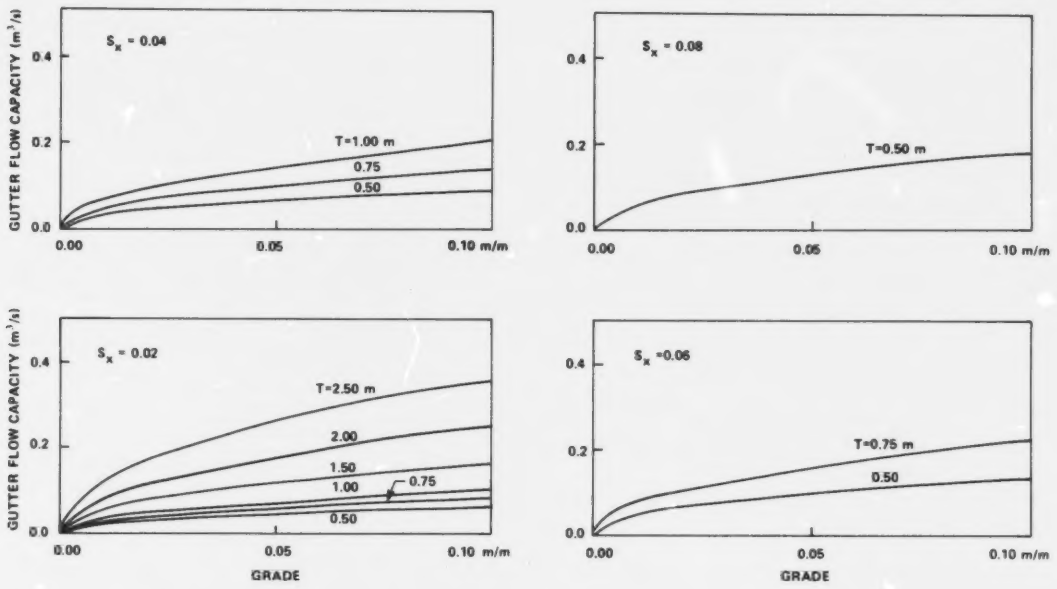


Figure 49/ Guter Flow Rating Curve: Curb & Gutter Type D

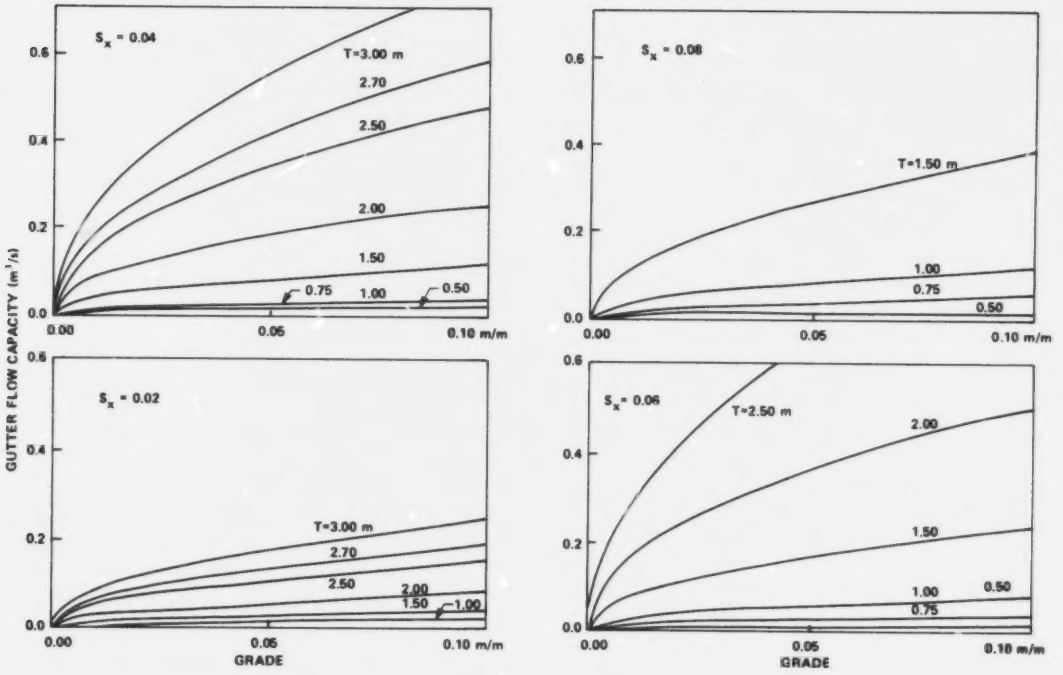


Figure 50/ Gutter Flow Capacity Curve: Curb Type E

8.3/ Runoff Peak Calculations

Assuming the applicability of the Rational Method, one can calculate the runoff peak from Equation 2, which was rearranged as follows:

$$Q_p\ (m^3/s) = 0.278 \times 10^{-6} \times Ci\ (mm/h) W(m) L(m) \quad (23)$$

where W and L are the width and length of the rectangular drainage area, respectively. The runoff coefficient C is typically taken as 0.95 and the average 10-year intensity i of 10 or 15 min duration (depending on the type of roadway) is obtained from the local rainfall data.

8.4/ Calculation of Inlet Spacings

Maintenance operations, such as cleanout of pipes, limit the spacing between inlets to about 150 m and such a restriction must be considered in inlet design.Note that this restriction does not apply to the first inlet because the sewer pipe starts only at the first inlet. One might argue, however, against excessively long drainage sections upstream of the first inlet because a large amount of debris may accumulate along a long flow path, be transported by the gutter flow, and eventually contribute to the blockage of the first inlet. This situation should be further reviewed by MTC and, if required, a restriction on the location of the first inlet imposed. Such a restriction could be less rigorous than that used for consecutive inlets for cleaning out of pipes (i.e., 150 m).

The calculation of the first inlet location (which is not affected by the spacing restrictions) is described first, followed by the calculations of consecutive spacings.

8.4.1/ Location of the First Inlet

To obtain the location of the first inlet on a continuous grade, it is assumed that this inlet is located at the point where the runoff peak flow from the drained area just reached the gutter flow capacity Q_{gT} corresponding to the design spread. Thus, one can substitute $Q_{gT} = Q_p$ into Equation 23 and obtain the following expression for inlet location:

$$L_1(m) = \frac{3.6 \times 10^6 \times Q_{gT}(m^3/s)}{Ci\ (mm/h)\ W(m)} \quad (24)$$

where Q_{gT} was determined from the design charts given in Figures 47-50.

At the first inlet, some fraction of the total flow is intercepted by the inlet and the remainder, referred to as a carryover flow Q_c, bypasses the inlet. This condition can be expressed as:

$$Q_{gT} = Q_{inT} + Q_c \quad (25)$$

where Q_{inT} is the inlet capacity for the spread T.

Past the inlet, the flow in the gutter increases because of the runoff contributions from the roadway.

8.4.2/ Spacing of Consecutive Inlets

The second inlet is installed at the point where the spread reaches the design value. The calculated spacing may or may not be subject to restrictions as discussed earlier. The case of no spacing restrictions is discussed first, followed by the case with a spacing restriction.

8.4.2.1/ Spacing of Consecutive Inlets without Spacing Restrictions

At the second inlet, the spread reaches the design value and, consequently, the gutter flow Q_g reaches the upper design limit (i.e., $Q_g = Q_{gT}$). After expressing the gutter flow as the sum of the carryover flow and runoff, $Q_{gT} = Q_c + Q_r$, and substituting for Q_{gT} from Equation 25, one obtains:

$$Q_r = Q_{inT} \quad (26)$$

After substituting Equation 26 into Equation 23 and solving for L_c, one obtains the following expression for inlet spacings:

$$L_c(m) = \frac{3.6 \times 10^6 \times Q_{inT}(m^3/s)}{Ci\ (mm/h)\ W(m)} \quad (27)$$

Note that the notation L_c is used here for the second and consecutive inlet spacings on a continuous grade and Q_{inT} is the upstream inlet capacity for a particular type of inlet and spread T. The inlet capacity can be determined from the inlet capacity curves given in Figures 8-11, 13-17, and 19. An example of the calculations described above follows.

Example No. 1

Given data:

$S_o = 0.003$ m/m, $S_x = 0.02$ m/m, n = 0.013, curb and gutter type A, inlet type DD-713-A, T = 2.5 m, and the drained area width W = 9.0 m. Hydrologic data — 10-year design storm, $t_c = 15$ min, $i_{15} = 100$ mm/h, C = 0.95.

Find:

Location of the first inlet and spacings of the consecutive inlets, there are no restrictions on spacings.

Step 1 — Determine the gutter flow capacity. For curb and gutter type A, $S_o = 0.003$ m/m, $S_x = 0.02$ m/m, and T = 2.5 m, one obtains from gutter capacity tables in the Appendix or from Figure 47:

$$Q_{gT} = 0.0567\ m^3/s$$

Step 2 — Calculate the first inlet location from Equation 24:

$$L_1 = \frac{3.6 \times 10^6 \times Q_{gT}}{C\ i\ W} = \frac{3.6 \times 10^6 \times 0.0567}{0.95 \times 100 \times 9} = 238.7\ m$$

Step 3 — Calculate the second and consecutive inlet spacings. First, determine the upstream (first) inlet capacity. For curb and gutter type A, $S_o = 0.003$ m/m, $S_x = 0.02$ m/m, and $T = 2.5$ m, and inlet DD-713-A, one reads from grate inlet capacity curves in the Appendix (or from Figure 8):

$$Q_{inT} = 0.0410 \text{ m}^3/\text{s}$$

The inlet spacing is then calculated from Equation 27 as:

$$L_c = \frac{3.6 \times 10^6 \times Q_{inT}}{C\,i\,W} = \frac{3.6 \times 10^6 \times 0.0410}{0.95 \times 100 \times 9} = 172.6 \text{ m}$$

Step 4 — Finalize the inlet spacing. Because there is no limit on inlet spacings, the first inlet is located 239 m from the upstream end and all the consecutive inlets are spaced at 173 m.

8.4.2.2/ Spacing of Consecutive Inlets with a Restriction

If an upper limit is imposed on inlet spacings, the designed drainage system may operate at pavement spreads below the design value. The calculation proceeds exactly as in the preceding Section 8.4.2.1, but the calculated consecutive spacing must meet the following condition:

$$L_c \leqslant L_{max} \qquad (28)$$

If the calculated L_c is less than L_{max}, it is implemented in the design and calculations are completed.

If the calculated L_c is larger than L_{max}, one simply adopts $L_c = L_{max}$ and, for most practical purposes, the calculations are completed. The designed drainage system will operate at smaller spreads than the design value. The operational parameters of such a drainage system can be calculated as shown below.

By adopting L_{max} for L_c ($L_{max} < L_c$), the gutter flow will be smaller than the design value Q_{gT} and the spread T_2 at the second inlet will be smaller than the design value T. Thus, one can write the following equation for a section immediately upstream of the second inlet:

$$Q_{gT2} = Q_{gT} - \Delta Q \qquad (29)$$

where $\Delta Q = 0.278 \times 10^{-6} \times C\,i\,W\,(L_c - L_{max})$.

Using the gutter capacity curves, one can determine spread T_2 corresponding to Q_{gT2} ($T_2 < T$). Finally, for T_2, one can determine from the inlet capacity curves, Q_{inT2}.

Thus, the carryover past the second inlet would be:

$$Q_{c2} = Q_{gT2} - Q_{inT2} \qquad (30)$$

The cases analysed indicate that L_3 would be slightly larger than L_c calculated for the second inlet. Consequently, one would adopt $L_3 = L_{max}$ and repeat the whole calculation starting with Equation 29. For such practical purposes, such iterative calculations demonstrated in the example below, have little value and can be omitted.

Example No.2

Given data:

$S = 0.003$ m/m, $S_x = 0.02$ m/m, $n = 0.013$, curb and gutter type A, inlet type DD-713-A, $T = 2.5$ m, and the drained area width $W = 9.0$ m. Hydrologic data — 10-year design storm, $t_c = 15$ min, $i_{15} = 100$ mm/h, $C = 0.95$.

Find:

Location of the first inlet and spacings of the consecutive inlets. The maximum allowable spacing is 150 m.

Step 1 — Determine the gutter flow capacity. For curb and gutter type A, $S_o = 0.003$ m/m, $S_x = 0.02$ m/m and $T = 2.5$ m, one obtains the following equation from gutter capacity tables in the Appendix (or from Figure 46):

$$Q_{gT} = 0.0567 \text{ m}^3/\text{s}$$

Step 2 — Calculate the first inlet location from Equation 24

$$L_1 = \frac{3.6 \times 10^6 \times Q_{gT}}{C\,i\,W} = \frac{3.6 \times 10^6 \times 0.0567}{0.95 \times 100 \times 9} = 238.7 \text{ m}$$

Step 3 — Calculate the second inlet spacing. First, determine the upstream (first) inlet capacity. For curb and gutter type A, $S_o = 0.003$ m/m, $S_x = 0.02$ m/m, $T = 2.5$ m, and inlet DD-713-A, one reads from grate inlet capacity curves in the Appendix (or from Figure 8):

$$Q_{inT} = 0.0410 \text{ m}^3/\text{s}$$

The inlet spacing is then calculated from Equation 27 as:

$$L_c = \frac{3.6 \times 10^6 \times Q_{inT}}{C\,i\,W} = \frac{3.6 \times 10^6 \times 0.0410}{0.95 \times 100 \times 9} = 172.6 \text{ m}$$

Because the spacing exceeds the maximum allowable value, one adopts for calculated the second inlet $L_2 = 150$ m.

Step 4 — Calculate the hydraulic characteristics for the second inlet (Q_{gT2}, T_2, Q_{inT2}, Q_{c2}). The gutter flow rate Q_{gT2} is calculated from Equation 29 as:

$$\begin{aligned} Q_{gT2} &= Q_{gT} - 0.278 \times 10^{-6} \times C\,i\,W\,(L_2 - L_{max}) \\ &= 0.0567 - 0.278 \times 10^{-6} \times 0.95 \times 100 \times 9 \times (172.6 - 150) \\ &= 0.0513 \text{ m}^3/\text{s} \end{aligned}$$

Using the capacity curve for gutter A, the spread T_2 is determined for $Q_g = 0.0513$ as $T_2 = 2.36$ m. For $T_2 = 2.36$ m, the inlet capacity is read from the inlet capacity tables (or Figure 8) as $Q_{in2} = 0.037$ m³/s. Finally, the carryover flow is expressed as:

$$Q_{c2} = Q_{gT2} - Q_{in2} = 0.513 - 0.0370 = 0.0143 \text{ m}^3/\text{s}$$

Step 5 — Calculate the third inlet spacing. This spacing is calculated from Equation 31 as:

$$L_3 = \frac{3.6 \times 10^6 (Q_{gT} - Q_{c2})}{C\, i\, W}$$

$$= \frac{3.6 \times 10^6 \times (0.0567 - 0.0143)}{0.95 \times 100 \times 9} = 178.5 \text{ m}$$

Step 6 — Calculate the hydraulic characteristics of the third inlet (Q_{gT3}, T_3, Q_{inT3}, Q_{c3}). The gutter flow rate is calculated from Equation 29 as:

$$Q_{gT3} = Q_{gT} - 0.000\,000\,278\, C\, i\, (L_3 - L_{max})$$
$$= 0.0567 - 0.000\,000\,278 \times 0.95 \times 100 \times 9 \times 28.5$$
$$= 0.0567 - 0.0068 = 0.0499 \text{ m}^3/\text{s}$$

Using the capacity curve for gutter A, the spread T_3 is determined, for $Q_{gT3} = 0.0499$, as $T_3 = 2.34$ m. For $T_3 = 2.34$ m, the inlet capacity is read from the inlet capacity tables (or Figure 8) as $Q_{in3} = 0.0367$ m^3/s. Finally, the carryover flow is expressed as:

$$Q_{c3} = Q_{gT3} - Q_{in3} = 0.0499 - 0.0367 = 0.132 \text{ m}^3/\text{s}$$

Step 7 — Calculate the fourth inlet spacing. This spacing is calculated from Equation 31 as:

$$L_4 = \frac{3.6 \times 10^6 (Q_{gT} - Q_c)_3}{C\, i\, W}$$

$$= \frac{3.6 \times 10^6 \times (0.0567 - 0.0132)}{0.95 \times 100 \times 9} = 183 \text{ m}$$

Step 8 — Finalize the inlet spacings. The first inlet is located 239 m from the upstream end and all the consecutive inlet spacings are 150 m.

The newly proposed design procedure for inlet spacing on a grade can be summarized as follows.

1/ The location of the first inlet is controlled by the gutter capacity corresponding to the design spread (see Equation 24). As suggested by MTC, no upper limit was placed on the length of the drained road section. The sewer pipe starts at the first inlet.

2/ The spacing of consecutive inlets is primarily controlled by the inlet capacity and maintenance requirements (sewer clean out). If the calculated spacing (Equation 27) is less than 150 m, the calculated value is adopted in the design and the system operates at the design spread. If the calculated spacing is larger than 150 m, the spacing of 150 m is adopted to satisfy the maintenance requirements. The designed system will operate at spreads below the design value.

8.5/ Inlets in a Sag

The design of inlets in a sag differs from that of inlets on a grade in several aspects. In particular, the entire inflow to the sag has to be intercepted and removed from the pavement to avoid uncontrolled water ponding and possible flood damages. Furthermore, inlets in a sag are more susceptible to blockage by debris than those installed on a grade and, consequently, the inlets in a sag are usually designed with a safety factor of two. In other words, the design flow must be conveyed by the inlet even if the inlet capacity is reduced by 50% as a result of blockage. A brief description of the proposed design procedure follows.

Firstly, one has to determine the inflow to the sag Q_{sag} which can be expressed as follows:

$$Q_{sag} = Q_r + Q_{cL} + Q_{cR} \qquad (32)$$

where Q_r is the runoff from the contribution area (calculated from Equation 23) and Q_{cL} and Q_{cR} are contributing carryover flows past the inlets, on a grade, on the left, and on the right side of the sag, respectively.

The inflow to the sag then has to be intercepted by inlets with capacities reduced by 50%, thus fulfilling the following condition:

$$N\,(0.5\, Q_{in\ sag}) \geqslant Q_{sag} \qquad (33)$$

where N is the required number of inlets, $Q_{in\ sag}$ is the inlet capacity in the sag (no blockage), and the numerical coefficient 0.5 expresses the reduction in the inlet capacity because of the blockage. The whole procedure is demonstrated in the example below.

Example No. 3

Given data:

Curb and gutter type A, inlet type DD-713-A, T = 2.5 m, $S_x = 0.02$ m/m, the drainage area width W = 9.5 m, $i_{15} = 150$ mm/h, C = 0.95, the last inlet on the left of the sag is 50 m from the sag, its carryover flow is $Q_{cL} = 0.0220$ m^3/s; the last inlet to the right of the sag is 40 m from the sag, its carryover flow is $Q_{cR} = 0.0250$ m^3/s.

Step 1 — Determine the inflow to the sag using Equation 32:

$$Q_{sag} = Q_r + Q_{cL} + Q_{cR}$$

where Q_r is the runoff from the contributing area 90 m long (50 + 40) and can be expressed from Equation 23 as follows:

$$Q_r = 0.278 \times 10^{-6} \times 0.95 \times 150 \times 9.5 \times 90 = 0.0339 \text{ m}^3/\text{s}$$

Thus, the sag inflow is:

$$Q_{sag} = 0.0339 + 0.0220 + 0.0250 = 0.0809 \text{ m}^3/\text{s}$$

Step 2 — Determine the sag inlet capacity. For spread T = 2.5 m, curb and gutter A, and crossfall $S_x = 0.02$ m/m, the depth of ponding at the curb equals the depth of gutter A, 0.05 m, plus the depth of ponding at the pavement edge — $T \times S_x$ 2.5 x 0.02 = 0.05 m. Thus, the total depth is 0.05 + 0.05 = 0.10 m. The capacity of the grate DD-713-A installed in a sag can be read from the table of sag inlet capacities, for d = 0.10 m, as $Q_{in\ sag} = 0.060$ m^3/s.

Step 3 — Determine the number of inlets required to intercept the sag inflow. Substituting for $Q_{in\ sag}$ and Q_{sag} into Equation 33 yields:

$$N(0.5 \times 0.0600) \geqslant 0.0809$$

$$N \times 0.0300 \geqslant 0.0809$$

For three inlets, one obtains

$$3 \times 0.0300 = 0.0900 \geqslant 0.0809$$

Thus, one needs to install three DD-713-A inlets to intercept the entire inflow to the sag and to maintain the desired design safety.

Additional information on placing of inlets in the sag is given in Section 8.6.1.

8.6/ Other Design Considerations

All the preceding calculations of inlet spacings were done for inlets installed on a continuous grade, or in a sag. Other aspects of inlet spacing design, which may call for more inlets than calculated for a constant grade, are discussed in this subsection. Such aspects include inlet locations, change in the grade, and a partial blockage of inlets.

8.6.1/ Inlet Locations

General rules for inlet locations, as given in a U.S. drainage manual [4] are presented below.

In general, inlets should be placed at all low points in the gutter grade and at intersections to prevent the gutter flow from crossing traffic lanes of the intersecting road. In urban locations, inlets are normally placed upgrade from pedestrian crossings to intercept the gutter flow before it reaches the crosswalk. Where pavement surfaces are warped, as at cross streets, ramps, or in transitions between superelevated and normal-sections, gutter flow should be picked up before the cross slope of the pavement begins to change in order to lessen water flowing across the roadway and to prevent icing.

In a sag vertical curve, three inlets should be placed —one at the low point and one on each side of this point where the grade elevation is approximately 0.06 m higher than that at the low point. The additional inlets furnish added capacity to allow for flow bypassing the upgrade inlets and provide a safety factor if the sag inlet becomes clogged. These inlets limit the deposition of sediment on the road in the sag. They also reduce flow arriving at the low point and thereby prevent ponding which would flood the road.

In a sag, combination inlets are very desirable. The curb opening provides a relief opening if the grate should become clogged.

Where a curbed roadway crosses a bridge, the gutter flow should be intercepted and not permitted to flow onto the bridge. This is particularly important where freezing temperatures occur.

Whenever possible, runoff from areas adjacent to the roadbed should be intercepted before reaching the pavement. This applies to water that would normally run onto the highway from side streets or from cut slopes and areas alongside the pavement. Street inlets are inefficient means for intercepting water and should not be used to intercept the runoff that could have been intercepted by more efficient structures such as open channels.

If runoff from adjacent areas cannot be intercepted (as may be the case in urban drainage), the runoff contribution from these areas should be added to the calculated runoff from the roadway.

8.6.2/ Change in the Grade

Changes in grade affect both the gutter flow and inlet capacities. The gutter flow will change from a uniform flow with a constant normal depth to a gradually varied flow with different normal depths for each grade. The transition from one normal depth to another may take various forms, depending on the steepness of both grades and the relative steepness of the two grades. The types of transitions include backwater curves, drawdown curves, and the hydraulic jump. A rigorous treatment of these transitions would be very complex and time consuming. Consequently, such flow transitions are neglected in accordance with current MTC design practices.

Assuming that the upper reach has a grade S_1 and the lower reach has a grade S_2, two cases can be considered. If $S_1 > S_2$, the gutter capacity, the carryover flow, and the inlet spacing in the upper reach will be larger than those in the low reach. Consequently, the carryover flow last from the upper reach has to be reduced, by reducing the inlet spacing calculated for S_1, to the level calculated for the lower reach.

The other case, $S_1 < S_2$, does not require any special considerations. If the last inlet spacing calculated for S_1 extends into the lower reach, this spacing may be retained. Such a procedure will increase the safety margin of the design, since the gutter capacity, the carryover flow, and the inlet spacing in the lower reach are larger than those in the upper reach.

To demonstrate various problems encountered in drainage design involving changes in the grade, Example No. 4 addressing this problem was included below. In this example, both an increase and reduction in the grade are considered. The computations are based on the previously-discussed formulae. It is assumed that the flow is uniform in individual reaches. Thus, the flow changes caused by changes in the grade do not propagate upstream or downstream.

Example No. 4

Given data:

Curb and gutter type B, inlet type DD-713-B, $S_x = 0.02$ m/m, design spread $T = 2.0$ m, the width of drainage area $W = 10$ m, the design intensity $i = 110$ mm/h, and $C = 0.95$. The total length of the drained road section is $l = 1600$ m and consists of three reaches with grades varying as follows: $S_1 = 0.01$ ($l_1 = 650$ m), $S_2 = 0.02$ ($l_2 = 500$ m), and $S_3 = 0.005$ ($l_3 = 450$ m).

Find:

Location of the first inlet and consecutive spacings, assuming no upper limit for spacings.

Step 1 — Determine the gutter flow capacity for the first reach. Using Figure 48, or the tables in the Appendix, one obtains for $S_1 = 0.01$ m/m, $S_x = 0.02$ m/m, and $T = 2.0$ m:

$$Q_{gT1} = 0.0617 \text{ m}^3/\text{s}$$

Step 2 — Calculate the first inlet location from Equation 24 as:

$$L_1 = \frac{3.6 \times 10^6 \times Q_{gT}}{C\,i\,w} = \frac{3.6 \times 10^6 \times 0.0617}{0.95 \times 110 \times 10.0} = 212.6 \text{ m}$$

Step 3 — Determine hydraulic characteristics of the first inlet. The inlet capacity is determined from Figure 11 or the tables in the Appendix (for $S_1 = 0.01$ m/m, $S_x = 0.02$ m/m, and $T = 2.0$ m) as:

$$Q_{inT1} = 0.0354 \text{ m}^3/\text{s}$$

The carryover flow can be expressed as:

$$Q_c = Q_{gT} - Q_{inT} = 0.0617 - 0.0354 = 0.0263 \text{m}^3/\text{s}$$

Step 4 — Calculate the second and consecutive inlet spacing from Equation 27 as:

$$L_c = \frac{3.6 \times 10^6 \times Q_{inT}}{C\,i\,w} = \frac{3.6 \times 10^6 \times 0.0354}{0.95 \times 110 \times 10.0} = 122 \text{ m}$$

Step 5 — Distribute inlets in the first reach ($l_1 = 650$ m):

Inlet No.	1	2	3	4
Total Length (m)	213	335	457	578

Step 6 — Calculate outflow from reach 1. This outflow equals the total runoff from reach 1, Q_{r1}, reduced for the sum of flows intercepted by inlets:

$$Q_{out1} = Q_{r1} - \sum_{j=1}^{N} Q_{inj}$$

where N is the total number of inlets.

The total runoff Q_{r1} can be calculated from Equation 23 considering the length of the drained area as $l_1 = 650$ m.

$$Q_{out1} = 0.278 \times 10^{-6} \times C \times i \times W \times l_1 - 4 \times Q_{inT}$$
$$= 0.278 \times 10^{-6} \times 0.95 \times 110 \times 10 \times 650 - 4 \times 0.0354$$
$$= 0.1888 - 0.1416 = 0.0472 \text{ m}^3/\text{s}$$

Step 7 — Determine the gutter flow capacity for the second reach. Using Figure 48, or the tables in the Appendix, one obtains for $S_2 = 0.02$ m/m, $S_x = 0.02$ m/m, and $T = 2.0$ m:

$$Q_{gT2} = 0.0873 \text{ m}^3/\text{s}$$

Step 8 — Determine the location of inlet No. 5. This inlet is located at the point where the gutter flow reaches the design value of 0.0873 m³/s. The gutter flow immediately upstream on inlet No. 5, Q_{gT2}, equals the outflow from the first reach plus the runoff generated between the beginning of the second reach and inlet No. 5. Thus:

$$Q_{gT2} = Q_{out1} + Q_{r5}$$
$$Q_{r5} = Q_{gT2} - Q_{out1}$$
$$= 0.0873 - 0.0472 = 0.0401 \text{ m}^3/\text{s}$$

The corresponding length, over which runoff Q_{r5} is generated, can be expressed from Equation 23 as:

$$L_5 = \frac{Q_{r5}}{0.278 \times 10^{-6} \times C \times i \times W}$$

$$= \frac{0.0401}{0.278 \times 10^{-6} \times 0.95 \times 110 \times 10} = 138 \text{ m}$$

Thus, inlet No. 5, is located 138 m from the beginning of the second reach or 788 m (= 650 + 138) from the upstream end of the drained road section.

It is of interest to note that where the grade increases in the downstream direction, one does not need to calculate the outflow from the upstream reach, because it is always smaller than the design gutter flow in the downstream reach. The calculation may proceed as follows.

Inlet No. 5 is placed at the point where the carryover past inlet No. 4 and the runoff generated between inlets Nos. 4 and 5 equals the design gutter flow Q_{gT2}. Thus:

$$Q_{c4} + Q_{r(4-5)} + Q_{gT2}$$

From this equation, one obtains $Q_{r(4-5)}$ as:

$$Q_{r(4-5)} = Q_{gT2} - Q_{c4} + 0.0873 - 0.0263 = 0.061 \text{ m}^3/\text{s}$$

The length, over which the runoff flow $Q_{r(4-5)}$ is generated, equals the fifth spacing and can be expressed from Equation 23 as:

$$L_5 = \frac{Q_{r(4-5)}}{0.278 \times 10^{-6} \times C \times i \times W}$$

$$= \frac{0.061}{0.278 \times 10^{-6} \times 0.95 \times 110 \times 10} = 210 \text{ m}$$

Thus, the fifth inlet is 210 m downstream from the fourth inlet, or 578 + 210 = 788 m from the upstream end of the drained road section.

Step 9 — Determine hydraulic characteristics of the fifth inlet. The inlet capacity is determined from Figure 11, or the tables in the Appendix, as:

$$Q_{inT2} = 0.0401 \text{ m}^3/\text{s}$$

The carryover flow can be expressed as:

$$Q_c = Q_{gT2} - Q_{inT2} = 0.0873 - 0.0401 = 0.0472 \text{ m}^3/\text{s}$$

Step 10 — Calculate the consecutive inlet spacing from Equation 27 as:

$$L_c = \frac{3.6 \times 10^6 \times Q_{inT2}}{C\,i\,W} = \frac{3.6 \times 10^6 \times 0.0401}{0.95 \times 110 \times 10} = 138.1 \text{ m}$$

Step 11 — Distribute inlets in the second reach (l_2 = 500 m):

Inlet No.	5	6	7
Total Length (m)	788	926	1064

Step 12 — Calculate outflow from reach 2 following the procedure outlined in Step 6:

Q_{out2}

$= 0.278 \times 10^{-6} \times C \times i \times W \times (l_1 + l_2) - 4 \times Q_{inT1} - 3 \times Q_{inT2}$
$= 0.278 \times 10^{-6} \times 0.95 \times 110 \times 10 \times (650 + 500)$
$- 4 \times 0.0354 - 3 \times 0.0401$
$= 0.3341 - 0.1416 - 0.1203$
$= 0.0722\ m^3/s$

Step 13 — Determine the gutter flow capacity for the third reach. Using Figure 48, or the tables in the Appendix, one obtains for $S^3 = 0.005$, $S_x = 0.02$, and T = 2.0:

$$Q_{gT3} = 0.0430\ m^3/s$$

The outflow from reach 2 exceeds the design gutter capacity (the corresponding spread in reach 3 would be 2.7 m, which exceeds the design value of 2.0 m). Consequently, the outflow from reach 2 must be reduced by adding another inlet.

Step 14 — Reduce the outflow from reach 2 by placing an inlet (No. 8) at the end of reach 2. For this purpose, determine the spread and inlet capacity at this point.

Considering the outflow from reach 2, $Q_{out2} = 0.0722\ m^3/s$, the corresponding spread can be determined from Figure 48, or the tables in the Appendix, as T = 1.77 m. For this spread, the inlet capacity is determined from Figure 11, or the tables in the Appendix, as:

$$Q_{in,T\,=\,1.77} = 0.0340\ m^3/s$$

The carryover past inlet No. 8 which equals the outflow from reach 2 is now:

$$Q_{out2} = 0.0722 - 0.0340 = 0.0382\ m^3/s$$

The outflow is acceptable, because it is smaller than the design gutter flow in reach 3.

Step 15 — Finalize the distribution of inlets in reach 2:

Inlet No.	5	6	7	8
Total Length (m)	788	926	1064	1150

Step 16 — Determine the location of the first inlet in reach 3 (i.e., inlet No. 9). The procedure is analogous to step No. 8.

Inlet No. 9 is located at the point where the outflow from reach 2 and the runoff generated between inlets Nos. 8 and 9 equal the design gutter capacity:

$$Q_{out2} + Q_{r(8-9)} = Q_{gT3}$$

From this equation, express $Q_{r(8-9)}$ as:

$Q_{r(8-9)} = Q_{gT3} - Q_{out2} = 0.0430 - 0.0382 = 0.0048\ m^3/s$

The length, over which runoff $Q_{r(8-9)}$ is generated, can be expressed from Equation 23 as:

$$L_{(8-9)} = \frac{Q_{r(8-9)}}{0.278 \times 10^{-6} \times C \times i \times W}$$

$$= \frac{0.0048}{0.278 \times 10^{-6} \times 0.95 \times 110 \times 10} = 16.5\ m$$

Thus, inlet No. 9 is placed 16.5 m downstream from the upstream end of reach 3.

Step 17 — Determine hydraulic characteristics of inlet No. 9. Using Figure 11, or the tables in the Appendix, the inlet capacity is determined as:

$$Q_{inT3} = 0.0265\ m^3/s$$

The carryover flow can be expressed as:

$Q_c = Q_{gT3} - Q_{inT3} = 0.0430 - 0.0265 = 0.0165\ m^3/s$

Step 18 — Calculate the consecutive inlet spacings from Equation 27 as:

$$L_c = \frac{3.6 \times 10^6 \times Q_{inT3}}{C\,i\,W} = \frac{3.6 \times 10^6 \times 0.0265}{0.95 \times 110 \times 10} = 91.3\ m$$

Step 19 — Finalize the distribution of inlets in reach 3:

Inlet No.	9	10	11	12	13
Total Length (m)	1167	1258	1349	1440	1531

Step 20 — Calculate the outflow Q_{out3} from reach 3 following the same procedure as in Steps 6 and 12:

Q_{out3}

$= \Sigma\, Q_r(l_1 + l_2 + l_3) - \Sigma\, Q_{in}l_1 - \Sigma\, Q_{in}l_2 - \Sigma\, Q_{in}l_3$
$= 0.278 \times 10^{-6} \times 0.95 \times 110 \times 10 \times (650 + 500 + 450)$
$- 4 \times 0.0354 - 3 \times 0.0401 - 0.0340 - 5 \times 0.0265$
$= 0.4648 - 0.1416 - 0.1203 - 0.0340 - 0.1325$
$= 0.0364\ m^3/s$

Depending on the circumstances, the outflow from reach 3 may have to be partly or fully intercepted.

Step 21 — Design summary and comments. The area is drained with 13 inlets spaced as follows (measured from the upstream end):

Inlet No.	1	2	3	4	5
Total Length (m)	213	335	457	578	788
Inlet No.	6	7	8	9	10
Total Length (m)	926	1064	1150	1167	1258
Inlet No.	11	12	13		
Total Length (m)	1349	1440	1531		

The outflow from the drained area is 0.0364 m^3/s. Note that this design was carried out without any upper limit on spacings, the two largest spacings were obtained for inlets Nos. 1 and 5 (213 and 210 m, respectively). The case with spacing restrictions was discussed in Example 2.

* * *

Special problems are encountered on a crest vertical curve where the grade is gradually increasing from zero, at the top, to some finite value. Again, one is facing a non-uniform flow problem in which the energy grade line and the water surface profile are not parallel to the channel bottom.

As discussed earlier, the location of the first inlet or drain is controlled by the gutter capacity corresponding to the design spread. Close to the top of the vertical curve, the grade is almost zerò and, consequently, the gutter flow capacity is negligible. Under these circumstances, the expressions for the first inlet location yield zero values. This physically meaningless solution follows from the uniform flow assumption that the energy grade line is parallel to the channel bottom. In real life, this is not the case. As water starts to accumulate on the pavement surface, the water surface will establish a finite gradient and water will flow in the direction of this gradient. It is possible to account for this fact in the proposed design procedure by adopting a minimum grade value (say, S_{min} = 0.003 m/m) for the top part of the vertical curve. This minimum grade is assumed only in the reach where the vertical curve is above the minimum grade. Downstream of the intersection of the vertical curve and the minimum grade, the grades averaged over the length of the inlet spacing may be used.

8.6.3/ Inlet Blockage

The inlet design procedure presented here is based on an assumption that inlets on a continuous grade are free of any blockage during the design events. The reasons for adopting this assumption, which deviates from the current practices, are the following.

- The probability of joint occurrence of a 10-year design storm used in highway drainage design and of an inlet blockage is relatively low because the high-intensity storms occur mostly in the summer months but inlet blockages are most frequent during the winter.
- Should an inlet blockage occur, the design spread would be exceeded and all the downstream inlets would operate at a higher-than-design capacity, thus compensating for the blocked inlet.
- Because the rare design events produce sharp, short-duration runoff peaks, road flooding and other inconveniences caused by inlet blockage would be of short duration.
- Finally, it is believed that the runoff peaks calculated by the Rational Method neglecting the surface storage are somewhat over estimated. Thus, there is some safety margin built into the proposed design procedure and this margin would tend to compensate for inlet blockage.

It is further recommended that the Ministry start to collect field data on inlet blockage. Such information would be useful for establishing joint probabilities of design events and inlet blockage.

Special attention has to be given to the blockage of inlets in a sag. Where inlet failures would result in large damages, an increased design safety is justified.

To illustrate the effects of a partial inlet blockage, the problem from Example No. 2 was recalculated assuming that the third inlet was partly blocked (operating at 50% of design capacity). As a result of this blockage, the design pavement spreads at inlet Nos. 4, 5, 6 and 7 were exceeded by not more than 0.26 m. At the eighth inlet, the spread returned back to the design value of 2.5 m.

8.7/ Discussion

The Ministry of Transportation and Communications intends to compare the new design procedure given here to the old one and to report the results elsewhere. Some limited observations on both procedures follow.

The main advantage of the new design procedure stems from the fact that the procedure is based on better information which was obtained from extensive full-scale tests of popular inlet grates. The design of inlet spacing was further simplified by devising design charts for individual gutters and grates. Although for clarity the gutter and inlet charts were presented separately in this report, the gutter flow rating curves and the corresponding inlet capacities could be plotted together in the MTC *Drainage Manual* [3].

It is expected that, for single inlets on a continuous grade, the new procedure will lead to reduction in the number of inlets in future work, thus resulting in lower capital and maintenance costs. In the case of twin inlets, the new procedure provides for a safer design. Old design charts indicate twin inlet capacities up to three times larger than those of single inlets. Such assumptions were refuted by the results given in this report (twin inlet capacities never exceeded those of single inlets by more than 70%).

The new design procedure should be also instrumental in better utilization of the sewer capacity. While the runoff from the total drained area enters the sewer system at some point (most likely in a sag), the distribution of runoff inflow into sewers is controlled by inlets. Consequently, sewer pipes need to be designed only for flow rates intercepted by inlets, realizing that some fraction of the runoff is conveyed by surface drainage. It should be reiterated that at some downstream point, all the surface runoff will enter the sewer system and a sufficient system capacity has to be provided at that point.

The question of inlet blockage deserves further attention. In the absence of well-documented field data on inlet blockage, the assumption of the degree of inlet blockage is a policy decision which may have far-reaching economic consequences. In this report, several arguments supporting the assumption of no blockage of inlets on a continuous grade were given. It was pointed out that the probability of inlet blockage was relatively low and that the blockage would cause minor inconvenience and hardly any damage. Under such circumstances, it is recommended to use full inlet capacities and reduce the cost of drainage. The situation is quite different in the case of inlets in a sag which are susceptible to clogging and their failure is likely to result in extensive inconvenience and damages, particularly if the sag is drained only by inlets and one deals with built-up areas. To reduce the risk of such damages, it is desirable to increase the safety margin of the inlet design by considering a 50% inlet blockage.

Another design aspect deserving further attention is the return period of the design event. As specified by MTC, a 10-year event was assumed throughout this report. Ideally, one should establish the most economical return period which would provide the minimum total drainage costs defined as:

Total Drainage Costs = Costs of Drainage Facilities + Costs of Drainage Failures

For short return periods, the facility costs are low and the failure costs are high and *vice versa*. The concept of minimum drainage costs is relatively new and will require further development.

The designer should have some appreciation of the drainage system response to higher-than-design runoff flows. This would be important only in areas where the costs of drainage failure are high (i.e., in sags drained only by inlets). It is suggested that under such circumstances, the designer should check the critical elements of the drainage scheme for such rare events as a 50-year or a 100-year storm.

9/ Design Procedure for Bridge Deck Drainage

The design of bridge deck drainage encompasses a large number of considerations including hydrological and hydraulic aspects, the type of structure, location, operation, and maintenance. The design procedure which follows concentrates on the hydrological and hydraulic aspects.

A survey of bridge deck drainage design practices indicated that these practices are largely based on experience and empiricism [17]. Very few agencies have detailed procedures for determining the drain spacing. Two such procedures are briefly reviewed below.

9.1/ Review of Selected Design Practices

9.1.1/ Idaho Procedure [4]

Having selected a drain position, the runoff peak flow Q (cfs) at this point is calculated from the Rational Method as follows:

$$Q = 0.000\,023\,A\,i \qquad (34)$$

where A is the deck area in ft^2, i is the rainfall intensity in in/h specified for six state districts. Using the calculated Q and a nomograph for flow in triangular channels (Figure 4), the flow spread is calculated. For a standard drain, it is assumed that all the road flow in a strip 1.5 ft wide (1.75 ft wide for grades smaller than 0.02 m/m), adjacent to the curb, enters the drain. The remaining flow, as determined from the earlier-mentioned nomograph, bypasses the drain. If this bypass is larger than 0.033 ft^3/s (0.001 m^3/s), another drain is located not less than 10 ft downstream from the first one. For most bridges, the maximum carryover flow from the last drain should not exceed 0.033 ft^3/s (about 0.001 m^3/s). The following deck drain locations are suggested — over medians, water or slope paving, in low points, in flat areas, and in areas with minimum carryover.

9.1.2/ California Procedure [17]

The runoff peak flow is computed from the Rational Method. Where precise values of rainfall intensity are not available, a 5 min rainfall intensity of 5 in/h is assumed. The runoff coefficient is taken as C = 1.0.

Drainage restrictions are such that no water is permitted to flow over joints, or the paving notch. The width of flow may vary, but should not encroach on the travelled way. Where the total flow interception is required, the drain width determines the maximum spread. The length of the drain grate varies depending on the flow velocity. Consequently, for a selected drain width, one determines the grate length from an empirical formula. The flow bypassing a given drain is added to the runoff quantity draining to the next following drain.

In summary, both procedures are fairly simplistic. The runoff peak flow is calculated from the Rational Method using typical values of the rainfall intensity. Actual drain capacities are not known; it is assumed that drains fully intercept road flows of a certain width. Typically, the drain positions are selected first and their inflow checked. If required, additional drains are placed shortly downstream.

Exhibit A

		Dimension
Description of the bridge structure and location (important for runoff disposal)		
Limiting factors or criteria (such as no flow over the joints or the pavement notch).		
Roadway width	W_p	m
Shoulder width	W_s	m
Curb type		
Total drained width	$W = W_p + W_s$	m
Total drained length	L_t	m
Grade (longitudinal slope)	S	m/m
Crossfall	S_x	m/m
Type (or types) of drains to be used		
Type of pavement and the corresponding Manning's roughness	n	—
Spread	T	m
Runoff coefficent	C	—
Concentration time	t_c	min
Return period of the design rainfall	R	years
The average design rainfall intensity for the concentration time t_c and the return period R (this value is read from a rainfall atlas)	i_{t_c}	mm/h
From the above data, one can calculate:		
The runoff peak flow (as a function of the length of the drained area)	Q_r	m^3/s
The maximum road flow capacity corresponding to spread T	Q_{gT}	m^3/s
Finally, the design is completed by calculating the following two parameters:		
Location of the first drain	L_1	m
Spacing for the consecutive drains	L_c	m

9.2/ Proposed Design Procedures

The newly proposed procedure has a number of distinct advantages over the procedures reviewed above. These advantages stem from the use of actual drain capacities, direct determination of drain spacings, and the computerization of the whole procedure. The computerized version simplifies the design and gives the designer a better appreciation of the drainage system behaviour. The general applicability of this procedure can be established only through extensive testing in actual design work.

A detailed description of the proposed design procedure follows.

9.2.1/ General Description

A notation sketch for the design procedure is given in Figure 51. For design of bridge deck drainage, the information shown in Exhibit A is given.

9.2.2/ Runoff Peak Calculation

As in the case of sewer inlets, the runoff peak flow is again calculated from the Rational Method in the following way:

$$Q_r = 0.278 \times 10^{-6} \times C\, i\, WL \qquad (35)$$

where Q_r is the runoff peak in m^3/s, C is the dimensionless runoff coefficient (typically taken as C = 0.95), i the average 10-year rainfall intensity of 10 to 15 min duration (depending on the type of roadway) in mm/h, and W and L are the width and length, respectively, of a rectangular drainage area in metres. For other details, see Chapter 2.

9.2.3/ Road Flow Calculation

The flow on the road surface with a crossfall may be described as an open-channel flow in a triangular channel. A modified Manning's equation for such a flow was presented in Chapter 3 in the following form:

$$Q_g = \frac{0.375}{n\, S_x} S^{1/2} d^{8/3} \qquad (36)$$

where Q_g is the road flow (in a triangular channel), n is the Manning's coefficient of roughness of the channel, S_x is the road (deck) crossfall, S is the grade, and d is the flow depth at the curb. Alternatively, one may use the nomograph given in Figure 4.

9.2.4/ Calculation of Drain Spacing

The calculation of drain spacing is based on the condition that the road flow spread immediately upstream of the drain will reach the design value. Such an approach is identical to that proposed earlier for sewer inlets in Chapter 8.

L_1 = FIRST DRAIN LOCATION
L_c = CONSECUTIVE DRAINS SPACING
W = DRAINED AREA WIDTH
T = PAVEMENT SPREAD
L_T = TOTAL DECK LENGTH

L_{dT} = DECK LENGTH MEASURED TO THE LAST DRAIN
Q_r = ROAD FLOW
L_r = REMAINING LENGTH PAST THE LAST DRAIN
Q_d = DRAIN DISCHARGE (CAPACITY)

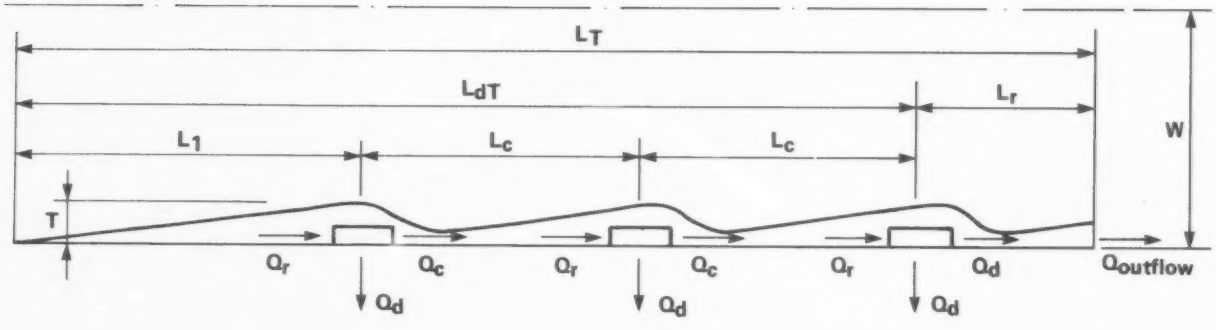


Figure 51/ Notation Sketch – Bridge Deck Drainage

First Drain Location — To obtain the location of the first drain on a continuous grade, the drain is assumed to be located at a point where the runoff peak flow from the drained area just reached the road flow Q_{gT} corresponding to the design spread:

$$Q_r = Q_{gT} \quad (37)$$

After substituting from Equations 35 and 36 into Equation 37, one obtains the following expression for the drain location measured from the upstream end of the drainage area:

$$L^1 = \frac{1.35 \times 10^6 \, S^{1/2} \, d^{8/3}}{C \, i \, W \, n \, S_x} \quad (38)$$

Equation 40 can be further rearranged by substituting $d = TS_x$:

$$L^1 = \frac{1.35 \times 10^6 \, S^{1/2} \, T^{8/3} \, S_x^{5/3}}{C \, i \, W \, n} \quad (39)$$

where d is the depth of flow at the curb, T is the spread, and S_x is the crossfall.

It is of interest to note that the location of the first drain does not depend on drain characteristics. It is controlled by the characteristics of the road flow.

At the first drain, some fraction of the road flow is intercepted by the drain and the remainder, referred to as a carryover flow, Q_c, bypasses the drain. This condition can be expressed as:

$$Q_{gT} = Q_{dT} + Q_c \quad (40)$$

where Q_{dT} is the drain discharge for the spread T.

Consecutive Drain Spacing — Past the first drain, the flow on the road starts to increase because of runoff contributions. The next drain is again installed at the point where the road flow reaches the design spread (i.e., $Q_g = Q_{gT}$). After expressing the road flow as $Q_g = Q_c + Q_r$ (where Q_c is the carryover flow from the previous upstream inlet) and substituting for Q_{gT} from Equation 40, one obtains:

$$Q_r = Q_{dT} \quad (41)$$

By substituting for Q_r from Equation 35 and solving for L_c, the consecutive spacing, one obtains the following final expression:

$$L_c = \frac{3.6 \times 10^6 \, Q_{dT}}{C \, i \, W} \quad (42)$$

The drain discharge Q_{dT}, which corresponds to the design spread T, can be determined from the regression equations (Equations 18 to 21) given earlier. In fact, one could substitute those equations into Equation 42 with the following results:

Drain SS 9-1A
SS 9-4A L_c

$$= \frac{3.6 \times 10^6 \, (0.000\,13 + 0.099\,1 \, TS_x - 0.118\,5 \, T^2 S_x^2)}{C \, i \, W} \quad (43a)$$

SS 9-2A
SS 9-2B L_c

$$= \frac{3.6 \times 10^6 \, (0.000\,39 + 0.268 \, TS_x - 0.440 \, T^2 S_x^2)}{C \, i \, W} \quad (43b)$$

SS 9-6A L_c

$$= \frac{3.6 \times 10^6 \, (0.000\,18 + 0.131 \, TS_x - 0.189 \, T^2 S_x^2)}{C \, i \, W} \quad (43c)$$

SS 9-8A
SS 9-8B L_c

$$= \frac{3.6 \times 10^6 \, (0.000\,9 + 0.53 \, TS_x - 1.47 \, T^2 S_x^2)}{C \, i \, W} \quad (43d)$$

The total number of drains N can be determined from the expression:

$$N = 1 + (L_T - L_1) / L_C \quad (44)$$

where L_t is the total length to be drained.

Typically, the calculated N will not be an integer. By rounding N down, there will be a length of the deck, L_r, contributing runoff past the last drain. Thus, the flow at the end of the drained area consists of the carryover past the last drain, plus the runoff generated over the length L_r:

$$Q_{outflow} = Q_c + Q_{rLr} \quad (45)$$

where Q_c can be calculated from Equation 40 and Q_{rLr} can be calculated from Equation 35 by substituting L_r. After making these substitutions, one obtains the following expression:

$$Q_{outflow} = \frac{0.375 \, S^{1/2} \, T^{8/3} \, S_x^{5/3}}{n} + 0.278 \times 10^{-6} \times C \, i \, W \, L_r - Q_{dT} \quad (46)$$

The designer then decides on the disposal of the outflow. If a full interception is required, more than one drain may be needed to achieve it.

Finally, the designer checks the calculated locations of drains and makes adjustments required to dispose of the drained water in suitable places.

The design procedure described above can be easily computerized using a programmable calculator. An example of a design program written for the Hewlett-Packard Calculator Model 9825 is given in Appendix B.

An example of a drain spacing design using the proposed procedure follows.

9.2.5/ Design Example

Given data:

$S = 0.02$ m/m; $S_x = 0.02$ m/m; $n = 0.013$; $W = 9.5$ m; $L_T = 1000$ m; $T = 1.5$ m; Drain SS 9-2A; $t_c = 15$ min; $i_{15} = 150$ mm/h; $C = 0.95$; the deck runoff should be fully intercepted.

Find:

The total number of drains.

Step 1 — Determine the first drain location from Equation 39.

$$L_1 = \frac{1.35 \times 10^6 S^{1/2} T^{8/3} S_x^{5/3}}{C\, i\, W\, n}$$

$$= \frac{1.35 \times 10^6\, 0.02^{1/2} \times 1.5^{8/3} \times 0.02^{5/3}}{0.95 \times 150 \times 9.5 \times 0.013} \times 47.1 \text{ m}$$

Step 2 — Determine the drain discharge corresponding to the spread of 1.5 m. The corresponding flow depth d can be calculated as:

$$d = TS_x = 1.5 \times 0.02 = 0.03 \text{ m}$$

The drain discharge is then calculated from Equation 19 as:

$$Q_{dT} = 0.000\,39 + 0.268 \times d - 0.44 \times d^2$$
$$= 0.000\,39 + 0.268 \times 0.03 - 0.44 \times 0.03^2$$
$$= 0.008\,0 \text{ m}^3/\text{s}$$

Check if the condition $Q_d < Q_{gT}$ is satisfied. For that purpose, calculate Q_{gT} from Equation 38 as:

$$Q_{gT} = \frac{0.375}{n\, S_x} S_o^{1/2} d^{8/3}$$

$$= \frac{0.375}{0.013 \times 0.02} \times 0.02^{1/2} \times 0.03^{8/3} = 0.0177 \text{ m}^3/\text{s}$$

Thus:

$$Q_{dT} = 0.008 < 0.0177 = Q_{gT}$$

Step 3 — Calculate the consecutive spacings from Equation 42:

$$L_c = \frac{3.6 \times 10^6\, Q_{dT}}{C\, i\, W} = \frac{3.6 \times 10^6 \times 0.008}{0.95 \times 150 \times 9.5} = 21.4 \text{ m}$$

Alternatively, Steps 2 and 3 could have been combined by using the more tedious Equation 43b with the same result.

$$L_c = \frac{3.6 \times 10^6\, (0.000\,39 + 0.268\, T\, S_x - 0.440\, T^2 S_x^2)}{C\, i\, W}$$

$$= \frac{3.6 \times 10^6\, (0.000\,39 + 0.268 \times 1.5 \times 0.02 - 0.440 \times 1.5^2 \times 0.02^2)}{0.95 \times 150 \times 9.5} = 21.4 \text{ m}$$

Step 4 — Calculate the number of drains needed from Equation 44:

$$N = 1 + (L_t - L_1)/L_c = 1 + (1000 - 47.1)/21.4 = 45.4$$

Assume $N = 45$ and calculate the drained length as:

$$L_{td} = L_1 + 44 \times L_c = 47.1 + 44 \times 21.4 = 988.7 \text{ m}$$

Step 5 — Calculate the outflow at the end of the bridge. Determine the length to be drained past the last drain.

$$L_r = L_t - L_{td} = 1000 - 988.7 + 11.3 \text{ m}$$

Substitute into Equation 46:

$$Q_{outflow}$$

$$= \frac{0.375\, S^{1/2}\, T^{8/3}\, S_x^{5/3}}{n} + 0.278 \times 10^{-6}\, C\, i\, WL_r - Q_{dT}$$

$$= \frac{0.375 \times 0.02^{1/2}\, 1.5^{8/3} \times 0.02^{5/3}}{0.013}$$

$$+ 0.278 \times 10^{-6} \times 0.95 \times 150 \times 9.5 \times 11.3 - 0.08$$

$$= 0.0140 \text{ m}^3/\text{s}$$

Step 6 — Determine the number of drains required to intercept the outflow.

Since $Q_{dT} = 0.0080$ m³/s (for $T = 1.5$ m), two drains are required to intercept the outflow. Thus, the total number of drains is $45 + 2 = 47$.

Step 7 — Finalize the design by checking the drain locations for suitable disposal of drained water.

9.2.6/ Other Design Considerations

Apart from the calculation of drain spacings, there are many other design considerations. Drains should not be located over a highway or railroad travelway because piping would be required. Locations over water, slope paving, or medians (with a suitable landing spot for water) are ideal. Higher density of drains than calculated may be required when placing drains in low points and flat areas, or when trying to avoid streams of water crossing the travelled roadway on bridges with superelevation and at the end of the centre divider curb, which is terminated short of the end of the bridge with superelevation.

The blockage of drains by debris may disrupt the operation of the bridge deck drainage. The problem of inlet blockage was discussed in Chapter 8. It was suggested that inlet blockage by debris and the resulting reduced capacity should be considered only in those cases where it could lead to serious damages and disruption of traffic. A good example of such a situation is the inlet in a sag. Similar recommendations apply to the bridge deck drains. It is believed that the problem of drain blockage can be prevented through regular inspection and maintenance procedures.

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Appendix A/ Data for Capacity Curves

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A4/ Ditch Inlet Capacity Curves . . . . . .
A4.1/ Data for Grates A-E, Grate Slope 1:1 . . . . .
A4.2/ Data for Grates A-E, Grate Slope 2:1 . . . . .

A5/ Bridge Deck Drain Capacity Curves. . . . . .
A5.1/ Data for Drains SS 9-1A, SS 9-2A, SS 9-2B, SS 9-4A, SS 9-6A, SS 9-8A, SS 9-8B . . . . .

A6/ Gutter Flow Rating Curves . . . . . .
A6.1/ Data for Curb and Gutter Type A . . . . . .
A6.2/ Data for Curb and Gutter Type B . . . . . .
A6.3/ Data for Curb and Gutter Type D . . . . . .
A6.4/ Data for Curb E (No Gutter) . . . . . .

A1 / Grate Inlet Capacity Curves (on Grade)

A1.1 / Data for Grate DD-713-A, Curb and Gutter Type A

Grate:	DD-713-A	Installation:	Undepressed, as per MTC specs.
Curb & Gutter:	Type A	Spread:	0.50 to 3.00 m
Grade:	0.003 to 0.100 m/m	Crossfall:	0.02 to 0.08 m/m
Source of Data:	Interpolations of experimental data	Graphical Presentation:	Figure 8

Crossfall S_x(m/m)	Spread T(m)	Grate Inlet Capacity (m^3/s) Grade S (m/m)							
		0.003	0.01	0.02	0.03	0.04	0.06	0.08	0.10
0.02	0.50	0.0055	0.0076	0.0094	0.0106	0.0114	0.0126	0.0129	0.0122
	0.75	0.0086	0.0126	0.0154	0.0172	0.0186	0.0198	0.0199	0.0191
	1.00	0.0125	0.0177	0.0216	0.0244	0.0260	0.0274	0.0270	0.0258
	1.50	0.0170	0.0252	0.0313	0.0352	0.0372	0.0383	0.0378	0.0364
	2.00	0.0274	0.0376	0.0445	0.0482	0.0497	0.0498	0.0489	0.0472
	2.50	0.0410	0.0506	0.0570	0.0597	0.0604	0.0602	0.0587	0.0572
	2.70	0.0452	0.0559	0.0616	0.0634	0.0639	0.0637	0.0625	0.0600
	3.00	0.0500	0.0613	0.0667	0.0687	0.0688	0.0675	0.0655	0.0636
0.04	0.50	0.0083	0.0127	0.0169	0.0201	0.0223	0.0241	0.0239	0.0222
	0.75	0.0136	0.0223	0.0300	0.0331	0.0357	0.0371	0.0362	0.0335
	1.00	0.0186	0.0295	0.0390	0.0436	0.0454	0.0469	0.0457	0.0432
	1.50	0.0338	0.0491	0.0586	0.0619	0.0628	0.0621	0.0599	0.0577
0.06	0.50	0.0107	0.0176	0.0236	0.0277	0.0306	0.0341	0.0358	0.0359
	0.75	0.0227	0.0315	0.0368	0.0405	0.0440	0.0477	0.0503	0.0494
	1.00	0.0339	0.0463	0.0537	0.0585	0.0610	0.0644	0.0632	0.0590
0.08	0.50	0.0151	0.0246	0.0318	0.0361	0.0390	0.0440	0.0460	0.0462
	0.75	0.0289	0.0419	0.0496	0.0533	0.0560	0.0603	0.0646	0.0676

A1.2/ Data for Grate DD-713-A, Curb and Gutter Type B

Grate:	DD-713-A	Installation:	Undepressed, as per MTC specs.
Curb & Gutter:	Type B	Spread:	0.50 to 3.00 m
Grade:	0.003 to 0.100 m/m	Crossfall:	0.02 to 0.08 m/m
Source of Data:	Interpolations of experimental data	Graphical Presentation:	Figure 10

Crossfall	Spread	Grate Inlet Capacity (m^3/s) Grade S (m/m)							
S_x(m/m)	T(m)	0.003	0.01	0.02	0.03	0.04	0.06	0.08	0.10
	0.50	0.0055	0.0076	0.0094	0.0106	0.0114	0.0126	0.0129	0.0122
	0.75	0.0083	0.0125	0.0160	0.0177	0.0191	0.0202	0.0204	0.0185
	1.00	0.0112	0.0166	0.0207	0.0232	0.0249	0.0263	0.0257	0.0242
0.02	1.50	0.0150	0.0250	0.0308	0.0339	0.0356	0.0367	0.0363	0.0350
	2.00	0.0249	0.0377	0.0434	0.0475	0.0487	0.0488	0.0474	0.0459
	2.50	0.0387	0.0501	0.0556	0.0590	0.0599	0.0586	0.0568	0.0555
	2.70	0.0410	0.0555	0.0592	0.0620	0.0625	0.0621	0.0607	0.0581
	3.00	0.0462	0.0615	0.0654	0.0676	0.0678	0.0671	0.0645	0.0627
	0.50	0.0085	0.0130	0.0169	0.0203	0.0220	0.0239	0.0237	0.0224
	0.75	0.0126	0.0209	0.0286	0.0324	0.0347	0.0368	0.0356	0.0320
	1.00	0.0169	0.0283	0.0379	0.0425	0.0444	0.0460	0.0445	0.0424
0.04	1.50	0.0303	0.0477	0.0572	0.0600	0.0611	0.0608	0.0583	0.0559
	2.00	0.0469	0.0700	0.0749	0.0767	0.0768	0.0750	0.0724	0.0681
	2.50	0.0624	0.0805	0.0872	0.0896	0.0887	0.0864	0.0820	0.0785
	0.50	0.0093	0.0165	0.0227	0.0266	0.0293	0.0328	0.0341	0.0342
	0.75	0.0211	0.0302	0.0356	0.0392	0.0419	0.0460	0.0477	0.0471
0.06	1.00	0.0335	0.0450	0.0518	0.0568	0.0592	0.0625	0.0610	0.0576
	1.50	0.0526	0.0729	0.0803	0.0842	0.0848	0.0836	0.0803	0.0754
	0.50	0.0163	0.0255	0.0324	0.0363	0.0384	0.0425	0.0444	0.0460
0.08	0.75	0.0280	0.0372	0.0443	0.0492	0.0511	0.0557	0.0590	0.0634
	1.00	0.0395	0.0496	0.0561	0.0602	0.0628	0.0672	0.0704	0.0744

A1.3/ Data for Grate DD-713-B, Curb and Gutter Type A

Grate:	DD-713-B	Installation:	Undepressed, as per MTC specs.
Curb & Gutter:	Type A	Spread:	0.50 to 3.00 m
Grade:	0.003 to 0.100 m/m	Crossfall:	0.02 to 0.08 m/m
Source of Data:	Interpolations of experimental data	Graphical Presentation:	Figure 9

Crossfall	Spread	Grate Inlet Capacity (m^3/s) Grade S (m/m)							
S_x(m/m)	T(m)	0.003	0.01	0.02	0.03	0.04	0.06	0.08	0.10
	0.50	0.0061	0.0090	0.0112	0.0125	0.0135	0.0143	0.0143	0.0133
	0.75	0.0092	0.0128	0.0168	0.0184	0.0197	0.0206	0.0202	0.0184
	1.00	0.0121	0.0172	0.0210	0.0237	0.0251	0.0262	0.0261	0.0244
0.02	1.50	0.0158	0.0237	0.0308	0.0339	0.0363	0.0377	0.0371	0.0344
	2.00	0.0245	0.0338	0.0421	0.0452	0.0467	0.0469	0.0453	0.0424
	2.50	0.0350	0.0467	0.0532	0.0553	0.0559	0.0558	0.0542	0.0520
	2.70	0.0380	0.0501	0.0573	0.0591	0.0595	0.0591	0.0577	0.0551
	3.00	0.0435	0.0561	0.0632	0.0645	0.0646	0.0640	0.0626	0.0600
	0.50	0.0080	0.0141	0.0193	0.0222	0.0243	0.0263	0.0260	0.0236
	0.75	0.0124	0.0209	0.0280	0.0322	0.0337	0.0350	0.0340	0.0319
0.04	1.00	0.0165	0.0271	0.0368	0.0416	0.0437	0.0447	0.0437	0.0408
	1.50	0.0295	0.0485	0.0571	0.0602	0.0610	0.0597	0.0573	0.0526
	0.50	0.0102	0.0188	0.0251	0.0277	0.0301	0.0331	0.0348	0.0335
0.06	0.75	0.0197	0.0291	0.0348	0.0382	0.0406	0.0444	0.0459	0.0453
	1.00	0.0309	0.0439	0.0507	0.0544	0.0570	0.0587	0.0582	0.0536
	0.50	0.0170	0.0257	0.0322	0.0359	0.0383	0.0409	0.0428	0.0438
0.08	0.75	0.0280	0.0387	0.0438	0.0470	0.0504	0.0549	0.0584	0.0597

A1.4/ Data for Grate DD-713-B, Curb and Gutter Type B

Grate: DD-713-B
Curb & Gutter: Type B
Grade: 0.003 to 0.100 m/m
Source of Data: Interpolations of experimental data

Installation: Undepressed, as per MTC specs.
Spread: 0.50 to 3.00 m
Crossfall: 0.02 to 0.08 m/m
Graphical Presentation: Figure 11

Crossfall	Spread	Grate Inlet Capacity (m^3/s)							
		Grade S (m/m)							
S_x(m/m)	T(m)	0.003	0.01	0.02	0.03	0.04	0.06	0.08	0.10
	0.50	0.0047	0.0073	0.0097	0.0110	0.0116	0.0125	0.0126	0.0122
	0.75	0.0079	0.0115	0.0142	0.0166	0.0177	0.0188	0.0193	0.0173
	1.00	0.0103	0.0143	0.0182	0.0209	0.0223	0.0233	0.0235	0.0218
0.02	1.50	0.0131	0.0226	0.0285	0.0315	0.0334	0.0345	0.0343	0.0321
	2.00	0.0229	0.0354	0.0401	0.0433	0.0439	0.0437	0.0427	0.0410
	2.50	0.0342	0.0461	0.0520	0.0542	0.0543	0.0536	0.0519	0.0494
	2.70	0.0364	0.0500	0.0558	0.0575	0.0577	0.0572	0.0555	0.0522
	3.00	0.0420	0.0549	0.0605	0.0619	0.0618	0.0605	0.0591	0.0571
	0.50	0.0069	0.0126	0.0170	0.0194	0.0215	0.0239	0.0236	0.0212
	0.75	0.0121	0.0210	0.0272	0.0304	0.0315	0.0324	0.0310	0.0281
	1.00	0.0156	0.0271	0.0345	0.0384	0.0399	0.0417	0.0403	0.0379
0.04	1.50	0.0275	0.0461	0.0537	0.0568	0.0576	0.0560	0.0529	0.0500
	2.00	0.0420	0.0642	0.0697	0.0708	0.0709	0.0698	0.0677	0.0636
	2.50	0.0575	0.0777	0.0806	0.0807	0.0797	0.0765	0.0733	0.0720
	0.50	0.0095	0.0154	0.0210	0.0242	0.0264	0.0280	0.0303	0.0301
	0.75	0.0186	0.0280	0.0328	0.0360	0.0393	0.0425	0.0444	0.0433
0.06	1.00	0.0302	0.0421	0.0479	0.0517	0.0539	0.0556	0.0549	0.0509
	1.50	0.0479	0.0621	0.0687	0.0712	0.0725	0.0713	0.0678	0.0625
	0.50	0.0131	0.0230	0.0295	0.0322	0.0350	0.0376	0.0383	0.0380
0.08	0.75	0.0271	0.0375	0.0422	0.0456	0.0489	0.0535	0.0565	0.0573
	1.00	0.0379	0.0504	0.0468	0.0609	0.0633	0.0681	0.0722	0.0743

A1.5/ Data for Grate DD-705, Curb and Gutter Type D

Grate: Type DD-705
Curb & Gutter: Type D
Grade: 0.003 to 0.100 m/m
Source of Data: Interpolations of experimental data

Installation: Undepressed, as per MTC specs.
Spread: 0.50 to 3.00 m
Crossfall: 0.02 to 0.08 m/m
Graphical Presentation: Figure 13

Crossfall	Spread	Grate Inlet Capacity (m^3/s)							
		Grade S (m/m)							
S_x(m/m)	T(m)	0.003	0.01	0.02	0.03	0.04	0.06	0.08	0.10
	0.50	0.0030	0.0048	0.0062	0.0077	0.0086	0.0097	0.0096	0.0093
	0.75	0.0062	0.0090	0.0122	0.0142	0.0157	0.0174	0.0176	0.0162
	1.00	0.0088	0.0138	0.0168	0.0188	0.0201	0.0215	0.0213	0.0202
	1.50	0.0163	0.0231	0.0282	0.0306	0.0318	0.0322	0.0308	0.0282
0.02	2.00	0.0246	0.0336	0.0389	0.0412	0.0412	0.0404	0.0384	0.0360
	2.50	0.0330	0.0456	0.0523	0.0538	0.0533	0.0502	0.0459	0.0413
	0.50	0.0067	0.0115	0.0149	0.0173	0.0185	0.0212	0.0230	0.0238
0.04	0.75	0.0132	0.0193	0.0242	0.0277	0.0294	0.0319	0.0333	0.0340
	1.00	0.0195	0.0253	0.0300	0.0328	0.0340	0.0365	0.0378	0.0380
	0.50	0.0188	0.0195	0.0251	0.0283	0.0298	0.0309	0.0304	0.0280
0.06	0.75	0.0224	0.0331	0.0393	0.0438	0.0459	0.0482	0.0476	0.0455
0.08	0.50	0.0205	0.0302	0.0343	0.0361	0.0375	0.0391	0.0381	0.0361

A1.6/ Data for Grate DD-713-A, Curb E (No Gutter)

Grate:	Type DD-713-A	Installation:	Undepressed, as per MTC specs.
Curb & Gutter:	Curb E (No Gutter)	Spread*:	0.50 to 3.00 m
Grade:	0.003 to 0.100 m/m	Crossfall:	0.02 to 0.08 m/m
Source of Data:	Interpolations of experimental data	Graphical Presentation:	Figure 15

Crossfall	Spread	Grate Inlet Capacity (m^3/s)							
		Grade S (m/m)							
S_x(m/m)	T(m)	0.003	0.01	0.02	0.03	0.04	0.06	0.08	0.10
0.02	0.50	0.0004	0.0007	0.0010	0.0014	0.0015	0.0016	0.0015	0.0012
	0.75	0.0010	0.0019	0.0028	0.0034	0.0038	0.0039	0.0034	0.0028
	1.00	0.0026	0.0040	0.0055	0.0067	0.0072	0.0075	0.0073	0.0065
	1.50	0.0063	0.0094	0.0129	0.0151	0.0160	0.0155	0.0142	0.0126
	2.00	0.0118	0.0171	0.0228	0.0262	0.0273	0.0263	0.0235	0.0207
	2.50	0.0188	0.0240	0.0309	0.0363	0.0394	0.0378	0.0333	0.0283
	2.70	0.0225	0.0282	0.0357	0.0420	0.0451	0.0428	0.0393	0.0348
	3.00	0.0297	0.0361	0.0440	0.0505	0.0540	0.0515	0.0472	0.0423
0.04	0.50	0.0012	0.0021	0.0030	0.0037	0.0044	0.0052	0.0049	0.0047
	0.75	0.0035	0.0054	0.0078	0.0097	0.0114	0.0134	0.0138	0.0126
	1.00	0.0076	0.0104	0.0140	0.0172	0.0200	0.0237	0.0236	0.0223
	1.50	0.0178	0.0237	0.0303	0.0356	0.0401	0.0460	0.0462	0.0434
	2.00	0.0313	0.0397	0.0488	0.0558	0.0620	0.0689	0.0685	0.0648
	2.50	0.0445	0.0542	0.0651	0.0739	0.0811	0.0889	0.0879	0.0841
	2.70	0.0501	0.0620	0.0726	0.0807	0.0878	0.0961	0.0959	0.0918
	3.00	0.0594	0.0726	0.0838	0.0921	0.0993	0.1080	0.1075	0.1023
0.06	0.50	0.0022	0.0039	0.0057	0.0069	0.0077	0.0084	0.0082	0.0077
	0.75	0.0064	0.0119	0.0162	0.0178	0.0182	0.0193	0.0194	0.0190
	1.00	0.0134	0.0214	0.0260	0.0290	0.0308	0.0321	0.0318	0.0302
	1.50	0.0313	0.0432	0.0498	0.0526	0.0544	0.0559	0.0555	0.0529
	2.00	0.0528	0.0634	0.0704	0.0734	0.0750	0.0765	0.0761	0.0744
	2.50	0.0715	0.0810	0.0867	0.0898	0.0913	0.0926	0.0921	0.0902
0.08	0.50	0.0031	0.0069	0.0094	0.0108	0.0119	0.0133	0.0127	0.0109
	0.75	0.0109	0.0175	0.0212	0.0235	0.0254	0.0265	0.0257	0.0233
	1.00	0.0194	0.0264	0.0318	0.0359	0.0387	0.0407	0.0400	0.0378
	1.50	0.0410	0.0556	0.0628	0.0665	0.0687	0.0698	0.0687	0.0665

*Measured from the foot of curb.

A1.7/ Data for Grate DD-713-B, Curb E (No Gutter)

Grate:	Type DD-713-B	Installation:	Undepressed, as per MTC specs.
Curb & Gutter:	Curb E (No Gutter)	Spread*:	0.50 to 3.00 m
Grade:	0.003 to 0.100 m/m	Crossfall:	0.02 to 0.08 m/m
Source of Data:	Interpolations of experimental data	Graphical Presentation:	Figure 4

		Grate Inlet Capacity (m^3/s)							
		Grade S (m/m)							
Crossfall S_x(m/m)	Spread T(m)	0.003	0.01	0.02	0.03	0.04	0.06	0.08	0.10
0.02	0.50	0.0004	0.0007	0.0010	0.0014	0.0015	0.0016	0.0015	0.0012
	0.75	0.0010	0.0019	0.0028	0.0034	0.0038	0.0039	0.0033	0.0026
	1.00	0.0026	0.0038	0.0052	0.0063	0.0068	0.0071	0.0069	0.0061
	1.50	0.0059	0.0088	0.0121	0.0142	0.0150	0.0146	0.0133	0.0118
	2.00	0.0110	0.0160	0.0213	0.0245	0.0255	0.0246	0.0220	0.0193
	2.50	0.0175	0.0223	0.0287	0.0338	0.0367	0.0352	0.0310	0.0263
	2.70	0.0210	0.0261	0.0331	0.0389	0.0418	0.0396	0.0364	0.0322
	3.00	0.0257	0.0325	0.0407	0.0468	0.0500	0.0477	0.0437	0.0392
0.04	0.50	0.0012	0.0020	0.0029	0.0036	0.0042	0.0050	0.0047	0.0045
	0.75	0.0033	0.0051	0.0074	0.0092	0.0109	0.0128	0.0131	0.0120
	1.00	0.0072	0.0098	0.0132	0.0162	0.0189	0.0224	0.0223	0.0210
	1.50	0.0167	0.0222	0.0283	0.0333	0.0375	0.0430	0.0432	0.0406
	2.00	0.0290	0.0368	0.0452	0.0517	0.0574	0.0638	0.0634	0.0600
	2.50	0.0410	0.0500	0.0600	0.0681	0.0748	0.0819	0.0810	0.0775
	2.70	0.0460	0.0569	0.0666	0.0740	0.0806	0.0882	0.0880	0.0787
	3.00	0.0540	0.0660	0.0762	0.0837	0.0903	0.0986	0.0978	0.0930
0.06	0.50	0.0021	0.0037	0.0054	0.0066	0.0073	0.0080	0.0078	0.0073
	0.75	0.0060	0.0112	0.0153	0.0168	0.0172	0.0182	0.0183	0.0179
	1.00	0.0125	0.0200	0.0251	0.0271	0.0288	0.0300	0.0297	0.0282
	1.50	0.0290	0.0400	0.0461	0.0487	0.0504	0.0518	0.0514	0.0490
	2.00	0.0485	0.0582	0.0646	0.0673	0.0688	0.0702	0.0698	0.0683
	2.50	0.0650	0.0736	0.0788	0.0816	0.0830	0.0842	0.0837	0.0820
0.08	0.50	0.0031	0.0065	0.0089	0.0102	0.0112	0.0125	0.0120	0.0103
	0.75	0.0102	0.0164	0.0198	0.0220	0.0237	0.0248	0.0240	0.0218
	1.00	0.0180	0.0244	0.0294	0.0332	0.0358	0.0377	0.0370	0.0350
	1.50	0.0380	0.0510	0.0576	0.0610	0.0630	0.0640	0.0630	0.0610

*Measured from the foot of curb.

A1.8/ Data for Grate DD-713-A, Curb and Gutter Type B, Twin/Single Capacity

Grate: Type DD-713-A
Curb & Gutter: Type B
Grade: 0.003 to 0.100 m/m
Source of Data: Interpolations of experimental data
Installation: Undepressed, as per MTC specs.
Spread: 0.50 to 3.00 m
Crossfall: 0.02 to 0.08 m/m
Graphical Presentation: Figure 17

Crossfall S_x(m/m)	Spread T(m)	Twin Inlet Capacity/Single Inlet Capacity (m^3/s) Grade S (m/m) 0.003	0.01	0.02	0.03	0.04	0.06	0.08	0.10
0.02	0.50	1.01	1.01	1.02	1.02	1.02	1.03	1.03	1.02
	0.75	1.02	1.03	1.03	1.04	1.05	1.06	1.06	1.06
	1.00	1.03	1.04	1.05	1.06	1.07	1.08	1.09	1.08
	1.50	1.04	1.06	1.08	1.10	1.12	1.15	1.17	1.16
	2.00	1.06	1.09	1.12	1.16	1.19	1.24	1.26	1.26
	2.50	1.08	1.12	1.17	1.21	1.25	1.32	1.36	1.35
	2.70	1.09	1.13	1.19	1.24	1.28	1.36	1.41	1.41
	3.00	1.11	1.16	1.22	1.28	1.34	1.43	1.48	1.47
0.04	0.50	1.02	1.03	1.04	1.05	1.06	1.07	1.07	1.06
	0.75	1.03	1.06	1.08	1.10	1.12	1.13	1.14	1.13
	1.00	1.04	1.08	1.12	1.16	1.18	1.20	1.20	1.19
	1.50	1.06	1.12	1.17	1.21	1.23	1.26	1.26	1.25
	2.00	1.09	1.16	1.22	1.27	1.30	1.34	1.35	1.34
	2.50	1.13	1.23	1.31	1.36	1.38	1.41	1.41	1.40
0.06	0.50	1.05	1.07	1.09	1.10	1.11	1.12	1.11	1.10
	0.75	1.07	1.10	1.13	1.15	1.16	1.17	1.17	1.16
	1.00	1.09	1.14	1.18	1.21	1.23	1.25	1.25	1.24
	1.50	1.12	1.18	1.23	1.27	1.29	1.31	1.31	1.30
0.08	0.50	1.06	1.09	1.12	1.13	1.14	1.14	1.14	1.13
	0.75	1.09	1.13	1.17	1.19	1.20	1.22	1.21	1.20
	1.00	1.12	1.18	1.23	1.27	1.29	1.31	1.31	1.30

A1.9/ Data for Grate DD-705, Curb and Gutter Type D, Twin/Single Capacity

Grate: Type DD-705 Twin/Single
Curb & Gutter: Type D
Grade: 0.003 to 0.100 m/m
Source of Data: Interpolations of experimental data
Installation: Undepressed, as per MTC specs.
Spread: 0.50 to 3.00 m
Crossfall: 0.02 to 0.08 m/m
Graphical Presentation: Figure 18

Crossfall S_x(m/m)	Spread T(m)	Twin Inlet Capacity/Single Inlet Capacity (m^3/s) Grade S (m/m) 0.003	0.01	0.02	0.03	0.04	0.06	0.08	0.10
0.02	0.50	1.00	1.01	1.01	1.02	1.02	1.02	1.01	1.01
	0.75	1.01	1.01	1.02	1.03	1.04	1.05	1.05	1.04
	1.00	1.01	1.02	1.05	1.06	1.08	1.10	1.09	1.08
	1.50	1.02	1.06	1.11	1.15	1.18	1.21	1.21	1.18
	2.00	1.07	1.10	1.20	1.26	1.29	1.33	1.34	1.33
	2.50	1.11	1.14	1.26	1.35	1.41	1.44	1.44	1.42
0.04	0.50	1.01	1.02	1.03	1.04	1.04	1.05	1.05	1.04
	0.75	1.03	1.07	1.10	1.12	1.13	1.14	1.13	1.12
	1.00	1.04	1.11	1.17	1.21	1.23	1.24	1.22	1.19
0.06	0.50	1.02	1.05	1.09	1.13	1.16	1.17	1.16	1.12
	0.75	1.08	1.13	1.19	1.25	1.30	1.34	1.33	1.31
0.08	0.50	1.08	1.12	1.17	1.21	1.25	1.30	1.30	1.29

A1.10/ Data for KWC Grate, Curb and Gutter Type B

Grate:	Type KWC	Installation:	Undepressed, as per MTC specs.
Curb & Gutter:	Type B	Spread:	0.50 to 3.00 m
Grade:	0.003 to 0.100 m/m	Crossfall:	0.02 to 0.08 m/m
Source of Data:	Interpolations of experimental data	Graphical Presentation:	Figure 20

Crossfall S_x(m/m)	Spread T(m)	Grate Inlet Capacity (m^3/s) Grade S (m/m)							
		0.003	0.01	0.02	0.03	0.04	0.06	0.08	0.10
0.02	0.50	0.0033	0.0069	0.0092	0.0101	0.0098	0.0083	0.0067	0.0047
	0.75	0.0057	0.0096	0.0124	0.0130	0.0129	0.0117	0.0098	0.0073
	1.00	0.0081	0.0122	0.0153	0.0162	0.0159	0.0147	0.0128	0.0106
	1.50	0.0121	0.0174	0.0213	0.0227	0.0230	0.0222	0.0203	0.0169
	2.00	0.0191	0.0254	0.0303	0.0328	0.0337	0.0334	0.0309	0.0266
	2.50	0.0268	0.0343	0.0391	0.0410	0.0413	0.0398	0.0364	0.0316
	2.70	0.0330	0.0398	0.0440	0.0454	0.0458	0.0436	0.0400	0.0356
	3.00	0.0392	0.0472	0.0513	0.0523	0.0523	0.0489	0.0447	0.0401
0.04	0.50	0.0066	0.0124	0.0167	0.0188	0.0201	0.0198	0.0174	0.0136
	0.75	0.0110	0.0178	0.0231	0.0257	0.0269	0.0269	0.0248	0.0211
	1.00	0.0153	0.0241	0.0294	0.0323	0.0336	0.0338	0.0322	0.0286
	1.50	0.0274	0.0365	0.0421	0.0451	0.0471	0.0478	0.0456	0.0411
	2.00	0.0461	0.0537	0.0587	0.0608	0.0610	0.0599	0.0566	0.0517
	2.50	0.0658	0.0737	0.0760	0.0751	0.0738	0.0702	0.0656	0.0597
0.06	0.50	0.0125	0.0174	0.0211	0.0231	0.0240	0.0238	0.0216	0.0177
	0.75	0.0175	0.0251	0.0306	0.0331	0.0343	0.0337	0.0306	0.0254
	1.00	0.0254	0.0355	0.0413	0.0438	0.0451	0.0430	0.0388	0.0331
	1.50	0.0430	0.0537	0.0597	0.0621	0.0622	0.0588	0.0527	0.0448
0.08	0.50	0.0141	0.0214	0.0262	0.0278	0.0280	0.0274	0.0263	0.0186
	0.75	0.0232	0.0322	0.0384	0.0411	0.0417	0.0398	0.0354	0.0291
	1.00	0.0348	0.0453	0.0522	0.0556	0.0561	0.0522	0.0464	0.0388

A2/ Grate Inlet Capacity Curves (In Sag)

A2.1/ Data for Common Curve for Single Grates DD-713-A, DD-713-B, DD-705, and KWC; Common Curve for Twin Inlets DD-713-A, DD-713-B, and DD-705

Grates: DD-713-A, DD-713-B, DD-705 (Single and Twin), KWC

Curb & Gutter: Type B

Depth of Ponding: 0.0 to 0.30 m

Crossfall: 0.02 to 0.06 m/m (results for various crossfalls were combined together)

Source of Data: Interpolations and Extrapolations of Experimental Data

Graphical Presentation: Figures 24-26

Depth of Ponding (m)	Inlet Capacity in Sag (m^3/s)		
	DD-713-A DD-713-B DD-705	Twin DD-713-A DD-713-B DD-705	KWC
0.00	0.0000	0.0000	0.0000
0.01	0.0004	0.0012	0.0004
0.02	0.0017	0.0028	0.0017
0.03	0.0040	0.0050	0.0040
0.04	0.0070	0.0098	0.0070
0.05	0.0110	0.0158	0.0110
0.06	0.0171	0.0256	0.0171
0.07	0.0250	0.0365	0.0250
0.08	0.0347	0.0530	0.0347
0.09	0.0464	0.0709	0.0464
0.10	0.0600	0.0908	0.0600
0.11	0.0726	0.1083	0.0726
0.12	0.0853	0.1256	0.0855
0.13	0.0971	0.1415	0.0975
0.14	0.1082	0.1548	0.1090
0.15	0.1184	0.1683	0.1220
0.16	0.1278	0.1820	0.1337
0.17	0.1363	0.1931	0.1444
0.18	0.1441	0.2111	0.1544
0.19	0.1510	0.2280	0.1638
0.20	0.1569	0.2440	0.1725
0.22	0.1671	0.2749	0.1891
0.24	0.1768	0.3067	0.2031
0.26	0.1860	0.3395	0.2151
0.28	0.1946	0.3727	0.2255
0.30	0.2027	0.4054	0.2340

A3/ Combination Inlet Capacity Curve (On Grade)

A3.1/ Data for Combination Inlet C-93

Grate:	Combination Inlet C-93	Installation:	Undepressed, as per MTC specs.
Curb & Gutter:	Type B	Spread:	0.50 to 3.00 m
Grade:	0.003 to 0.100 m/m	Crossfall:	0.02 to 0.08 m/m
Source of Data:	Interpolations of experimental data	Graphical Presentation:	Figure 29

Crossfall S_x(m/m)	Spread T(m)	Combination Inlet Capacity (m^3/s) Grade S (m/m)							
		0.003	0.01	0.02	0.03	0.04	0.06	0.08	0.10
0.02	0.50	0.0035	0.0060	0.0081	0.0093	0.0101	0.0100	0.0097	0.0086
	0.75	0.0053	0.0085	0.0112	0.0128	0.0137	0.0146	0.0136	0.0116
	1.00	0.0065	0.0111	0.0144	0.0164	0.0174	0.0180	0.0172	0.0150
	1.50	0.0102	0.0168	0.0207	0.0227	0.0239	0.0245	0.0233	0.0210
	2.00	0.0169	0.0245	0.0288	0.0316	0.0332	0.0343	0.0329	0.0303
	2.50	0.0245	0.0313	0.0356	0.0378	0.0391	0.0398	0.0383	0.0361
	2.70	0.0287	0.0349	0.0387	0.0411	0.0422	0.0429	0.0415	0.0389
	3.00	0.0342	0.0400	0.0429	0.0448	0.0459	0.0461	0.0449	0.0423
0.04	0.50	0.0054	0.0106	0.0150	0.0178	0.0194	0.0206	0.0199	0.0182
	0.75	0.0094	0.0156	0.0210	0.0242	0.0260	0.0278	0.0273	0.0255
	1.00	0.0135	0.0207	0.0267	0.0300	0.0318	0.0331	0.0319	0.0295
	1.50	0.0248	0.0319	0.0360	0.0381	0.0393	0.0395	0.0377	0.0346
	2.00	0.0376	0.0444	0.0478	0.0494	0.0501	0.0497	0.0472	0.0436
	2.50	0.0501	0.0575	0.0611	0.0621	0.0622	0.0607	0.0568	0.0520
0.06	0.50	0.0085	0.0148	0.0190	0.0213	0.0225	0.0232	0.0219	0.0196
	0.75	0.0154	0.0255	0.0302	0.0315	0.0321	0.0311	0.0285	0.0250
	1.00	0.0270	0.0357	0.0391	0.0398	0.0394	0.0375	0.0343	0.0300
	1.50	0.0432	0.0491	0.0498	0.0492	0.0480	0.0446	0.0400	0.0346
0.08	0.50	0.0104	0.0186	0.0230	0.0250	0.0259	0.0263	0.0250	0.0228
	0.75	0.0202	0.0300	0.0355	0.0371	0.0377	0.0364	0.0330	0.0283
	1.00	0.0304	0.0398	0.0434	0.0451	0.0450	0.0433	0.0396	0.0353

A4 / Ditch Inlet Capacity Curves

A4.1 / Data for Grates A-E, Grate Slope 1:1

Grate Width: 1.00 m (for other widths, multiply the capacities below by the actual width)

Grate Slope: 1:1

Inlet Type: A - E

Graphical Presentation: Figure 33

Water Depth (m)	Inlet Capacity (m^3/s)				
	A (DD-710)	B	C	D	E
0.02	0.0253	0.0203	0.0228	0.0202	0.0218
0.04	0.0370	0.0310	0.0337	0.0308	0.0326
0.06	0.0508	0.0439	0.0469	0.0438	0.0456
0.08	0.0669	0.0592	0.0623	0.0592	0.0607
0.10	0.0852	0.0767	0.0798	0.0767	0.0782
0.12	0.1056	0.0964	0.0995	0.0963	0.0979
0.14	0.1283	0.1184	0.1215	0.1183	0.1196
0.16	0.1532	0.1427	0.1456	0.1427	0.1438
0.18	0.1802	0.1693	0.1719	0.1692	0.1700
0.20	0.2095	0.1981	0.2003	0.1979	0.1984
0.25	0.2923	0.2800	0.2806	0.2798	0.2790
0.30	0.3889	0.3761	0.3748	0.3759	0.3735
0.35	0.4992	0.4863	0.4828	0.4859	0.4814
0.40	0.6233	0.6107	0.6040	0.6103	0.6032
0.45	0.7612	0.7493	0.7390	0.7488	0.7387
0.50	0.9128	0.9019	0.8875	0.9016	0.8881

A4.2 / Data for Grates A-E, Grate Slope 2:1

Grate Width: 1.00 m (for other widths, multiply the capacities below by the actual width)

Grate Slope: 2:1

Inlet Type: A - E

Graphical Presentation: Figure 34

Water Depth (m)	Inlet Capacity (m^3/s)				
	A (DD-710)	B	C	D	E
0.02	0.0103	0.0090	0.0088	0.0095	0.0101
0.04	0.0196	0.0180	0.0178	0.0186	0.0191
0.06	0.0321	0.0300	0.0294	0.0305	0.0310
0.08	0.0477	0.0451	0.0443	0.0456	0.0459
0.10	0.0660	0.0634	0.0623	0.0637	0.0637
0.12	0.0875	0.0845	0.0830	0.0849	0.0844
0.14	0.1122	0.1088	0.1069	0.1090	0.1080
0.16	0.1398	0.1361	0.1337	0.1361	0.1345
0.18	0.1703	0.1666	0.1637	0.1660	0.1639
0.20	0.2037	0.2000	0.1966	0.1992	0.1963
0.25	0.3011	0.2971	0.2920	0.2952	0.2899
0.30	0.4172	0.4133	0.4061	0.4098	0.4016
0.35	0.5520	0.5485	0.5393	0.5435	0.5316
0.40	0.7058	0.7032	0.6910	0.6958	0.6796
0.45	0.8788	0.8767	0.8615	0.8668	0.8459
0.50	1.0703	1.0695	1.0509	1.0570	1.0302

A5/ Bridge Deck Drain Capacity Curves

A5.1/ Data for Drains SS 9-1A, SS 9-2A, SS 9-2B, SS 9-4A, SS 9-6A, SS 9-8A, SS 9-8B

Drains: SS 9-1A, SS 9-2A, SS 9-2B, SS 9-4A, SS 9-6A, SS 9-8A, SS 9-8B

Source of Data: Interpolations and Extrapolations of Experimental Data

Grades & Crossfalls: The curves below were derived for grades from 0.005 m/m to 0.07 m/m, and crossfalls from 0.02 m/m to 0.06 m/m

Graphical Presentation: Figures 37, 39 and 41

Depth of Flow (m)	Bridge Deck Drain Capacity (m^3/s)			
	SS 9-1A SS 9-4A	SS 9-2A SS 9-2B	SS 9-6A	SS 9-8A SS 9-8B
0.000	0.0000	0.0000	0.0000	0.0000
0.005	0.0006	0.0017	0.0008	0.0028
0.010	0.0011	0.0030	0.0015	0.0057
0.015	0.0016	0.0043	0.0021	0.0083
0.020	0.0021	0.0056	0.0027	0.0109
0.025	0.0025	0.0068	0.0033	0.0132
0.030	0.0030	0.0080	0.0039	0.0155
0.035	0.0035	0.0092	0.0045	0.0176
0.040	0.0039	0.0104	0.0051	0.0197
0.045	0.0044	0.0116	0.0057	0.0218
0.050	0.0048	0.0127	0.0063	0.0237
0.060	0.0057	0.0149	0.0074	0.0274
0.070	0.0065	0.0170	0.0084	0.0308
0.080	0.0073	0.0190	0.0095	0.0339
0.090	0.0081	0.0209	0.0104	0.0367
0.100	0.0089	0.0228	0.0114	0.0392
0.110	0.0096	0.0245	0.0123	0.0414
0.120	0.0103	0.0262	0.0132	0.0433
0.130	0.0110	0.0278	0.0140	0.0450
0.140	0.0117	0.0293	0.0148	0.0463
0.150	0.0123	0.0307	0.0156	0.0473

A6/ Gutter Flow Rating Curves

A6.1/ Data for Curb and Gutter Type A

Curb & Gutter: Type A
Grade: 0.003 to 0.10 m/m
Spread: 0.0 to 3.0 m (measured from the gutter edge on the pavement side)
Crossfall: 0.02 to 0.08 m/m
Method of Computation: Manning's Equation integrated over the width of channel; n = 0.013
Graphical Presentation: Figure 47

Crossfall S_x(m/m)	Spread (m)	Flow Depth(m)	Gutter Flow Rate (m^3/s) Grade S (m/m) 0.003	0.010	0.020	0.030	0.040	0.060	0.080	0.100
	0.00	0.050	0.0055	0.0100	0.0141	0.0173	0.0200	0.0244	0.0282	0.0315
	0.50	0.060	0.0092	0.0168	0.0237	0.0290	0.0335	0.0411	0.0474	0.0530
	0.75	0.065	0.0119	0.0217	0.0307	0.0376	0.0434	0.0531	0.0614	0.0686
	1.00	0.070	0.0153	0.0279	0.0395	0.0484	0.0559	0.0685	0.0790	0.0884
0.02	1.50	0.080	0.0248	0.0452	0.0639	0.0783	0.0904	0.1107	0.1278	0.1429
	2.00	0.090	0.0383	0.0699	0.0989	0.1211	0.1399	0.1713	0.1978	0.2211
	2.50	0.100	0.0567	0.1034	0.1463	0.1791	0.2069	0.2534	0.2925	0.3271
	2.70	0.104	0.0655	0.1195	0.1691	0.2071	0.2391	0.2928	0.3381	0.3780
	3.00	0.110	0.0804	0.1465	0.2077	0.2544	0.2937	0.3597	0.4154	0.4644
	0.50	0.070	0.0141	0.0258	0.0365	0.0447	0.0516	0.0633	0.0730	0.0817
	0.75	0.080	0.0213	0.0389	0.0551	0.0674	0.0779	0.0954	0.1101	0.1231
0.04	1.00	0.090	0.0309	0.0565	0.0798	0.0978	0.1129	0.1383	0.1597	0.1785
	1.50	0.110	0.0587	0.1071	0.1515	0.1855	0.2142	0.2624	0.3030	0.3387
	1.68*	0.125	0.0882	0.1610	0.2277	0.2788	0.3220	0.3943	0.4553	0.5091
	0.50	0.080	0.0202	0.0369	0.0521	0.0638	0.0737	0.0903	0.1042	0.1165
	0.75	0.095	0.0334	0.0609	0.0861	0.1055	0.1218	0.1492	0.1723	0.1926
0.06	1.00	0.110	0.0514	0.0939	0.1327	0.1626	0.1877	0.2299	0.2655	0.2968
	1.25*	0.125	0.0750	0.1370	0.1937	0.2372	0.2739	0.3355	0.3874	0.4331
	0.50	0.090	0.0272	0.0497	0.0703	0.0861	0.0994	0.1218	0.1406	0.1572
0.08	0.75	0.110	0.0478	0.0872	0.1234	0.1511	0.1745	0.2137	0.2467	0.2759
	0.94*	0.125	0.0964	0.1249	0.1767	0.2164	0.2499	0.3061	0.3534	0.3951

Drainage channel filled to top of curb.

A6.2/ Data for Curb and Gutter Type B

Curb & Gutter: Type B
Grade: 0.003 to 0.10 m/m
Spread: 0.0 to 3.0 m (measured from the gutter edge on the pavement side)
Crossfall: 0.02 to 0.08 m/m
Method of Computation: Manning's Equation integrated over the width of channel; n = 0.013
Graphical Presentation: Figure 48

Crossfall	Spread	Flow	Gutter Flow Rate (m^3/s) Grade S (m/m)							
S_x(m/m)	(m)	Depth(m)	0.003	0.010	0.020	0.030	0.040	0.060	0.080	0.100
	0.00	0.050	0.0045	0.0081	0.0115	0.0141	0.0163	0.0200	0.0230	0.0258
	0.50	0.060	0.0076	0.0138	0.0195	0.0239	0.0276	0.0338	0.0391	0.0437
	0.75	0.065	0.0099	0.0181	0.0255	0.0313	0.0361	0.0442	0.0511	0.0571
	1.00	0.070	0.0129	0.0235	0.0333	0.0408	0.0471	0.0577	0.0666	0.0745
0.02	1.50	0.080	0.0214	0.0390	0.0552	0.0676	0.0781	0.0956	0.1104	0.1235
	2.00	0.090	0.0338	0.0617	0.0873	0.1069	0.1234	0.1511	0.1745	0.1951
	2.50	0.100	0.0508	0.0928	0.1312	0.1607	0.1856	0.2273	0.2625	0.2934
	2.70	0.104	0.0591	0.1079	0.1525	0.1868	0.2157	0.2642	0.3051	0.3411
	3.00	0.110	0.0731	0.1335	0.1888	0.2312	0.2670	0.3270	0.3775	0.4221
	0.50	0.070	0.0117	0.0214	0.0303	0.0371	0.0428	0.0525	0.0606	0.0677
	0.75	0.080	0.0180	0.0328	0.0464	0.0568	0.0656	0.0803	0.0927	0.1037
	1.00	0.090	0.0264	0.0482	0.0682	0.0835	0.0964	0.1181	0.1364	0.1525
0.04	1.50	0.110	0.0513	0.0937	0.1326	0.1623	0.1875	0.2296	0.2651	0.2964
	2.00	0.130	0.0889	0.1624	0.2296	0.2812	0.3247	0.3977	0.4592	0.5134
	2.50*	0.150	0.1414	0.2581	0.3650	0.4470	0.5162	0.6322	0.7300	0.8162
	0.50	0.080	0.0168	0.0307	0.0434	0.0532	0.0614	0.0752	0.0868	0.0971
	0.75	0.095	0.0282	0.0515	0.0729	0.0892	0.1030	0.1262	0.1457	0.1629
0.06	1.00	0.110	0.0441	0.0805	0.1138	0.1394	0.1610	0.1971	0.2276	0.2545
	1.50	0.140	0.0918	0.1675	0.2369	0.2902	0.3351	0.4104	0.4739	0.5298
	1.67*	0.150	0.1130	0.2064	0.2919	0.3575	0.4128	0.5055	0.5837	0.6526
	0.50	0.090	0.0227	0.0415	0.0587	0.0719	0.0830	0.1016	0.1173	0.1312
	0.75	0.110	0.0405	0.0739	0.1044	0.1279	0.1477	0.1809	0.2089	0.2336
0.08	1.00	0.130	0.0655	0.1196	0.1691	0.2071	0.2391	0.2928	0.3381	0.3781
	1.25*	0.150	0.0989	0.1805	0.2552	0.3126	0.3610	0.4421	0.5105	0.5707

*Drainage channel filled to top of curb.

A6.3/ Data for Curb and Gutter Type D

Curb & Gutter: Type D
Grade: 0.003 to 0.10 m/m
Spread: 0.0 to 2.5 m (measured from the gutter edge on the pavement side)
Crossfall: 0.02 to 0.08 m/m
Method of Computation: Manning's Equation integrated over the width of channel; n = 0.013
Graphical Presentation: Figure 49

Crossfall S_x(m/m)	Spread (m)	Flow Depth(m)	Gutter Flow Rate (m^3/s) Grade S (m/m)							
			0.003	0.010	0.020	0.030	0.040	0.060	0.080	0.100
0.02	0.00	0.050	0.0063	0.0115	0.0162	0.0199	0.0230	0.0281	0.0325	0.0363
	0.50	0.060	0.0105	0.0192	0.0272	0.0333	0.0385	0.0471	0.0544	0.0609
	0.75	0.065	0.0136	0.0248	0.0351	0.0429	0.0496	0.0607	0.0701	0.0784
	1.00	0.070	0.0174	0.0317	0.0449	0.0550	0.0635	0.0778	0.0898	0.1004
	1.50	0.080	0.0278	0.0508	0.0718	0.0879	0.1015	0.1243	0.1436	0.1605
	2.00	0.090	0.0426	0.0777	0.1099	0.1346	0.1555	0.1904	0.2100	0.2458
	2.50*	0.100	0.0625	0.1140	0.1613	0.1975	0.2281	0.2793	0.3225	0.3606
0.04	0.50	0.070	0.0162	0.0296	0.0419	0.0513	0.0593	0.0726	0.0838	0.0937
	0.75	0.080	0.0244	0.0445	0.0629	0.0771	0.0890	0.1090	0.1259	0.1407
	1.00	0.090	0.0352	0.0643	0.0909	0.1113	0.1285	0.1574	0.1818	0.2032
	1.25*	0.100	0.0491	0.0896	0.1267	0.1552	0.1792	0.2194	0.2534	0.2833
0.06	0.50	0.080	0.0232	0.0424	0.0600	0.0735	0.0848	0.1039	0.1200	0.1341
	0.75	0.095	0.0384	0.0700	0.0990	0.1213	0.1401	0.1716	0.1981	0.2215
	0.83*	0.100	0.0446	0.0814	0.1151	0.1410	0.1628	0.1994	0.2302	0.2574
0.08	0.50	0.090	0.0315	0.0575	0.0813	0.0996	0.1150	0.1409	0.1627	0.1819
	0.63*	0.100	0.0424	0.0774	0.1094	0.1340	0.1547	0.1895	0.2188	0.2447

*Drainage channel filled to top of curb.

A6.4/ Data for Curb E (No Gutter)

Curb & Gutter: Curb E (No Gutter)
Grade: 0.003 to 0.10 m/m
Spread: 0.0 to 3.0 m (measured from the foot of the curb)
Crossfall: 0.02 to 0.08 m/m
Method of Computation: Manning's Equation integrated over the width of channel; n = 0.013
Graphical Presentation: Figure 50

Crossfall S_x(m/m)	Spread (m)	Flow Depth(m)	Gutter Flow Rate (m^3/s) Grade S (m/m)							
			0.003	0.010	0.020	0.030	0.040	0.060	0.080	0.100
0.02	0.00	0.010	0.0004	0.0007	0.0010	0.0012	0.0013	0.0016	0.0019	0.0021
	0.75	0.015	0.0011	0.0020	0.0028	0.0034	0.0040	0.0049	0.0056	0.0063
	1.00	0.020	0.0023	0.0043	0.0060	0.0074	0.0085	0.0105	0.0121	0.0135
	1.50	0.030	0.0069	0.0126	0.0178	0.0218	0.0252	0.0309	0.0356	0.0399
	2.00	0.040	0.0149	0.0271	0.0384	0.0470	0.0543	0.0665	0.0768	0.0858
	2.50	0.050	0.0270	0.0492	0.0696	0.0852	0.0984	0.1206	0.1392	0.1556
	2.70	0.054	0.0331	0.0604	0.0855	0.1047	0.1209	0.1482	0.1709	0.1911
	3.00	0.060	0.0438	0.0800	0.1132	0.1386	0.1601	0.1961	0.2264	0.2531
0.04	0.50	0.020	0.0012	0.0022	0.0030	0.0037	0.0043	0.0053	0.0061	0.0068
	0.75	0.030	0.0035	0.0063	0.0090	0.0110	0.0127	0.0156	0.0179	0.0201
	1.00	0.040	0.0075	0.0136	0.0193	0.0237	0.0273	0.0335	0.0387	0.0432
	1.50	0.060	0.0220	0.0403	0.0570	0.0698	0.0806	0.0987	0.1139	0.1274
	2.00	0.080	0.0475	0.0868	0.1227	0.1503	0.1735	0.2125	0.2454	0.2743
	2.50	0.100	0.0862	0.1573	0.2225	0.2725	0.3146	0.3854	0.4449	0.4975
	2.70	0.108	0.1058	0.1932	0.2732	0.3345	0.3863	0.4731	0.5463	0.6108
	3.00	0.120	0.1410	0.2558	0.3618	0.4431	0.5117	0.6266	0.7236	0.8090
0.06	0.50	0.030	0.0023	0.0043	0.0060	0.0074	0.0085	0.0104	0.0120	0.0135
	0.75	0.045	0.0069	0.0126	0.0178	0.0217	0.0251	0.0307	0.0355	0.0397
	1.00	0.060	0.0148	0.0270	0.0382	0.0468	0.0541	0.0662	0.0765	0.0855
	1.50	0.090	0.0437	0.0797	0.1127	0.1381	0.1594	0.1952	0.2254	0.2520
	2.00	0.120	0.0940	0.1717	0.2428	0.2973	0.3434	0.4205	0.4856	0.5429
	2.50	0.150	0.1705	0.3113	0.4402	0.5392	0.6226	0.7625	0.8805	0.9844
0.08	0.50	0.040	0.0038	0.0069	0.0098	0.0120	0.0138	0.0170	0.0196	0.0219
	0.75	0.060	0.0112	0.0204	0.0289	0.0353	0.0408	0.0500	0.0577	0.0645
	1.00	0.080	0.0241	0.0440	0.0622	0.0761	0.0879	0.1077	0.1243	0.1390
	1.50	0.120	0.0710	0.1296	0.1833	0.2245	0.2592	0.3174	0.3666	0.4098
	1.875*	0.150	0.1287	0.2350	0.3323	0.4070	0.4700	0.5756	0.6647	0.7431

*Drainage channel filled to top of curb.

Appendix B/ Computer Program for Design of Bridge Deck Drainage

For Hewlett-Packard Programmable Calculator Model 9825 (Program Listing and a Sample Run)

B.1 / Program for Design of Bridge Deck Drainage Based on a Design Spread

```
0: fxd 4
1: prt " ********** "
2: spc                                  ---------- Enter Data ----------
3: enp "Dr. Area Length [m] ?", L       L = length of the area to be drained
4: enp "Dr. Area Width [m] ?", W        W = width of the area to be drained
5: enp "Design Spread [m] ?", T         T = design spread
6: enp "Crossfall?", F                  F = crossfall
7: enp "Grade?", G                      G = grade
8: enp "Mannings n?", N                 N = Manning's n
9: enp "Runf. Coeff?", C                C = runoff coefficient
10: enp "Rainf. Int. [mm/h] ?" I        I = design rainfall intensity (typically t = 15 min, 10-year)
11: enp "Drain No.?", D                 D = drain type; 1 = SS 9-1A, 2 = SS 9-2A, 3 = SS 9-2B, 4 = SS9-4A, 5 = SS 9-6A
12: enp "Drain Clogging [%] ?", R       R = percent of drain blocked by debris (= percent of capacity reduction)
13: spc                                 ----------
14: prt "    ****    "
15: F*T→r1                              r1 = flow depth in a triangular channel corresponding to spread T and crossfall F
16: if D=2;gto 21
17: if D=3;gto 21                       } Depending on the drain type, directs control to an appropriate equation.
18: if D=5;gto 23
19: .00013+.0991*r1-.1185*r1↑2→Q        Q_d 1,4 = f(r1) drain capacity, Eq. 1; drains SS 9-1A and 4A
20: gto 24
21: .00039+.2684*r1-.4396*r1↑2→Q        Q_d 2,3 = f(r1) drain capacity, Eq. 2; drains SS 9-2A and 2B
22: gto 24
23: .00018+.131*r1-.1894*r1↑2→Q         Q_d 5 = f(r1) drain capcity, Eq. 3; drain SS 9-6A
24: .375*√G*r1↑2.667/(N*F)→r3           r3 = road flow rate (triangular channel, grade G, crossfall F). If drain capacity
                                             exceeds road flow, set equal to road flow.
25: if Q>r3;r3→Q
26: r3-Q→P                              P = carryover flow = road flow – drain capacity
27: (1-R/100)*Q→Q                            Calculates reduced drain capcity for various degrees of blockage.
28: 1350000*√G*T↑2.667*F↑1.667/
    C*I*W*N)Dr11                        r11 = L_1 = first drain spacing, Eq. 10
29: 3597000*Q/(C*I*W)→r12               r12 = L_c = consecutive drains spacing, Eq. 13
30: 0→J
31: J+1→J                               J = a counter
32: r11+J*r12→r13                            – keeps adding drains till the total length to be drained is just exceeded
33: if r13<L;gto 31
34: J-1→K
35: r11+K*r12→r14                       r14 = L_dT = deck length measured to the last drain
36: L-r14→r15                           r15 = L_r = remaining length past last drain
37: .000000278*C*I*W*r15→r16            r16 = Q_pLr = runoff from remaining length
38: r16+P→r17                           r17 = Q_outflow
```

```
39: if D=2; gto 45
40: if D=3; gto 47
41: if D=4; gto 49
42: if D=5; gto 51
43: prt "Drain No. SS 9-1A"
44: gto 52
45: prt "Drain No. SS 9-2A"
46: gto 52
47: prt "Drain No. SS 9-2B"
48: gto 52
49: prt "Drain No. SS 9-4A"
50: gto 52
51: prt "Drain No. SS 9-6A"
52: prt "Qroad[m³/s]", r3
53: fxd 2
54:prt "Max. Spread [m]", T
55: fxd 3
56: prt "Max. Depth [m]", r1
57: fxd 4
58: prt "Qdrain [m³/s]", Q
59: prt "Qcarryov. [m³/s]", P
60: spc 1
61: fxd 1
62: prt "1st Spacing [m]", r11
63: prt "2nd Spacing [m]", r12
64: fxd 0
65: prt "Number of Drains", J
66: fxd 1
67: spc 1
68: prt "Drained L [m]=", r14
69: prt "Remainder [m]", r15
70: fxd 4
71: prt "Rem. Runoff [m³/s]", r16
72: prt "Outflow [m³/s]", r17
73: spc
74: prt "    **********    "
75: stp
76: end
*953
```

--- Directs control for printout

---------- Printout ----------

--- Identifies the drain type

r3 = road flow rate

T = design spread

r1 = flow depth at the curb (corresponds to spread T)

Q_d = drain capacity (for spread T)

P = Q_c = carryover flow rate

r11 = L_1 = first drain spacing

r12 = L_c = consecutive drains spacing

J = total number of drains

r14 = L_{dT} = deck length measured to the last drain

r15 = L_r = remaining length past the last drain

r16 = Q_{pLr} = runoff from the remaining length

r17 = $Q_{outflow}$

END

B.2/ Sample Run of Program for Bridge Deck Drainage Design

INPUT DATA	OUTPUT
**********	****
Dr. Area Length [m] ?	Drain No. SS 9-2A
1000	Qroad [m^3/s]
Dr. Area Width [m] ?	0.0125
9	Max. Spread [m]
Design Spread [m] ?	1.50
1.5	Max. Depth [m]
Crossfall?	0.030
.02	Qdrain [m^3/s]
Grade?	0.0080
.01	Qcarryov. [m^3/s]
Mannings n?	0.0045
.013	1st Spacing [m]
Runf. Coeff?	52.7
.95	2nd Spacing [m]
Rainf. Int. [mm/h] ?	33.9
100	Number of Drains
Drain No.?	28
2	Drained L [m] =
Drain Clogging [%] ?	966.7
0	Remainder [m]
	33.3
	Rem. Runoff [m^3/s]
	0.0079
	Outflow [m^3/s]
	0.0124
	*******